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HYDRAULIC-FILL DAMS*

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With Discussion by Messrs. C. E. Curtis, Richard D. Chase, G. A. M. Elliott, J. M. Howells, D. C. Henny, A. T. Goldbeck, George L. Dillman, H. B. Muckleston, H. F. Dunham, William H. Burr, M. M. O'Shaughnessy, Arthur E. Morgan, H. deB. Parsons, George Sidney Binckley, Charles H. Paul, John E. Field, J. Albert Holmes, H. W. Swanitz, Joel D. Justin, and Allen Hazen.

Synopsis.

The hydraulic method of dam construction has great advantages, but too many dams built by it have failed.

As often built, the center of the dam is composed of a clay core completely water-tight, but very wet as a result of the method of construction, and so completely water-tight that it is incapable of permitting its own drainage.

This soft core in the center of the dam exerts the pressure of a liquid of high gravity on the toes and tends to push them out. This pressure seems to have been responsible for most of the failures of hydraulic-fill dams.

The method of placing the core commonly used results in an exceedingly fine-grained material, desirable from the standpoint of a water-tight dam; but it may be that extreme tightness is secured at the expense of stability.

To use the hydraulic method of dam construction successfully, it is necessary either to increase the toes until they are large and heavy

* Presented at the meeting of May 5th, 1920.

enough successfully to resist the full fluid pressure of the core, or else to handle the core material so that the finest particles are wasted, leaving only the larger ones, and in that way increasing the grain size to a point where drainage can be secured, while on the other hand remaining sufficiently impervious for practical purposes.

Methods of testing are described and an effort is made to analyze the conditions of stability.

The hydraulic-fill method of dam construction grew out of hydraulic mining. It was natural that a process conspicuously successful in mining on an enormous scale, and which moved material at very low unit costs, should find other applications.

The late James D. Schuyler and J. M. Howells, Members, Am. Soc. C. E., were pioneers in the application of hydraulic methods to dam construction. Mr. Schuyler presented a paper* to the Society in 1906 setting forth in a most admirable way the peculiarities of the method and its application to actual dams.

The method is certain to have an important place in future dam construction. It has two fundamental advantages: First, in sorting out the fine particles and placing them in the center of the dam, thus insuring complete water-tightness; and, second, in the use of power in place of muscle to the greatest extent, so that the labor cost and the animal cost of construction are reduced to a minimum.

It is sometimes possible to place material by the hydraulic process at much smaller cost than by any other method.

On the other hand, several hydraulic dams have not proved to be stable. Failures, usually during construction, have caused great losses, and their occurrence has been discouraging.

There may have been more than one cause of failure, but one cause has certainly predominated. In the interior of the dams there have been masses of clay and other fine-grained material, that have not consolidated to the point of stability. Instead of forming an integral part of a solid dam, these masses have remained in almost liquid form, dividing the dams and tending to disrupt them. In this respect, hydraulic-fill dams are on a different basis from other earth dams.

If the hydraulic method of dam construction is to be used successfully, this element of weakness must be eliminated.

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* *Transactions*, Am. Soc. C. E., Vol. LVIII (1907), p. 196.

There seem to be two promising ways of eliminating it. The first and most natural one is to increase the size of the dam until the solid parts forming the toes are amply strong to resist the full fluid pressure of the unstable core. The second consists in selecting material and handling it in a way to secure drainage and consolidation of the core, so that the whole dam will act as one solid mass.

If core material can be fully drained and consolidated, there is no reason why a dam built by the hydraulic method on a section that is suitable for an earth dam built dry should not be safe. If drainage and consolidation of the core are not secured, however, safety can only be reached by making each toe large enough to stand the full pressure of the liquid core, and in that case, in order to be safe, a dam built by the hydraulic method must have flatter slopes and a larger section than is otherwise needed for an earth dam.

It is interesting to note that Clemens Herschel, Past-President, Am. Soc. C. E., in discussing Mr. Schuyler's paper in 1906, made the point that a larger section would be required for stability; Mr. Schuyler took exception to this, and stated that in his experience there was no reason for building a dam of greater dimensions because the material was placed in it by the hydraulic method.

The advantages and economies of the hydraulic method are such that it will often be good business to use it, even though the required volume of the dam should be greater. If work can be built at half the cost per cubic yard, twice as many cubic yards can be placed for the same money.

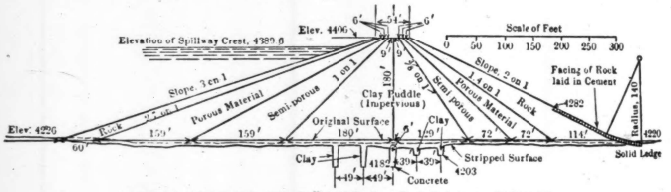
The number of failures with hydraulic-fill dams growing out of this fundamental condition of fluid cores and inadequate toes has been such as to make it clear that this condition of instability has not been recognized in the designs. Mr. Schuyler's excellent paper contains no discussion of this point. Looking at it from the standpoint of thirteen years afterward, it is easy to see how this condition could have been overlooked and to see, also, how some of the early failures were attributed to other causes.

Now, however, enough experience has been gained so that a better analysis may be made. To insure success with future hydraulic dams, this analysis must be thorough. It is the writer's thought that it will be possible to make it in a way to secure as high a degree of safety in hydraulic-fill dams as is required, and is customary, in other engineer-

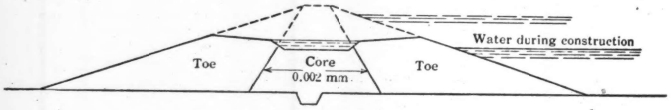
ing structures. It may be that we do not yet have all the necessary data; but, if not, the best way to find what is lacking is by using what we have.

Character of Core Material.

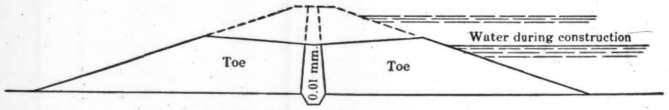
The first point to be considered is that, in many or perhaps most hydraulic-fill dams, the core material is so fine in grain size that it is incapable of drainage. By that is meant that it is incapable of drainage within a reasonable length of time. The lower parts of the core do not become stable before the upper parts are placed upon them.



CROSS-SECTION OF DAM NO. 2 AT NECAXA, MEXICO.
SHOWING
DIMENSIONS, CUT-OFF TRENCHES AND THEORETICAL DISTRIBUTION OF MATERIALS.
FIG. 1



TYPICAL SECTION OF HYDRAULIC FILL
DAM DURING CONSTRUCTION.
FIG. 2



ILLUSTRATING PROPOSED SECTION WITH
NARROW COARSE-GRAINED CORE.
FIG. 3

In general, all the material placed in hydraulic-fill dams may be divided into two classes, namely, toe material and core material. Mr. Schuyler presented a diagram of materials divided into several classes, the coarsest being found at the outside and the finest in the middle (Fig. 1*). Practical experience does not show that the material is graded in quite that way. There is some grading in the toes and the average size of particles near points of delivery of material near the outer slopes is

* Fig. 1 is reproduced from *Transactions*, Am. Soc. C. E., Vol. LVIII (1907), p. 243.

greater than away from those points and near the core; but, practically speaking, this gradation is not very important. Ordinarily, all the toe material is of coarse particles when first placed. Afterward, passage of silt-laden water from the operation of hydraulicking partly fills the voids with fine material, but the process stops itself before they are all filled. The toes drain readily and all the stability naturally inherent in the stock is obtained.

The core may be defined as that part of the dam deposited from water in the central pool. In the writer's experience, core material is homogeneous. At any level there is no appreciable difference in grain size between the middle of the core and its farthest extremities.

The principles determining the size of particles deposited in the core, or not deposited, are the same as those that apply in a sedimentation basin of a water-works plant. The larger the area of the pool in proportion to the quantity of water that goes through it, the more complete will be the subsidence, and the smaller will be the limit of size of particles retained.*

Most hydraulic-fill dams have been built with good-sized pools in the center during construction (Fig. 2), in which sedimentation has been fairly complete. The writer has examined samples of the core materials from a number of dams. The samples are rather surprisingly alike in grain size. They ordinarily contain particles in large quantities down to a limit between 0.001 and 0.002 mm. (or sometimes 0.003 mm.) in diameter. Generally speaking, all smaller particles have been carried away by the escaping water and have not remained in the core.

It should be stated that the determination of the exact size of these small particles is by no means as precise as the determination of the size of grains of filter sand. The results are reached by the microscope with a micrometer and are to be considered as roughly approximate only. It has been the intention to use a basis as nearly as possible comparable to that adopted for the larger particles in sand analysis, which is to use as the diameter of any particle the diameter of a sphere of equal volume. Following this rule the width, rather than the length, of small particles is measured.

Core material then, as a matter of observation, consists of particles from this size up to the size of the smallest particles retained in the

* "On Sedimentation," by Allen Hazen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 45.

toes. In a general way, this size may be taken as 0.03 mm., or ten to fifteen times as great as the limit of size of particles that are retained in the core.

Speaking of core materials in the terms that are used in describing sand; it may be stated that the effective size of core material is ordinarily about 0.002 mm. This size is so small that one who is not accustomed to microscopic work has little conception of it. For comparison, it may be stated that ordinary filter sand has an effective size of from 0.3 to 0.4 mm., while dune sand, being the finest material usually spoken of as sand, has effective sizes ranging from 0.15 to 0.20 mm.

Without attempting precision, and using round figures, the core material is one-hundredth of the size of dune sand. This means (assuming that the laws of flow and the laws of capillarity applying to the finest sands apply also to these still finer materials) that, other things being equal, the drainage of core material will take ten thousand times as long as corresponding drainage of fine sand. Fine dune sand would drain as much in an hour as core material in a year. It means, further, that the height to which water can be held by capillarity is ten thousand times as great as the height at which it is held by fine sand. If fine sand will hold water by capillarity to a height of an inch, as it will, then core material like that shown by the samples will hold water by capillarity to much more than the full height of any dam.

The reason why this core material does not drain is therefore clear. It seals itself up and becomes practically water-tight.

With the dimensions existing in dams that have been built, assuming that the laws of flow known to apply to the finest sands apply also to the still finer materials, it will take years for the excess of moisture to drain out, as it must do before the solidification of the material is possible.

As a practical proposition, drainage is not possible with such fine materials. By that is meant such prompt and complete drainage as will result in solidification during the construction of the dam.

Construction records and samples show approximately what happens. Core material of the fineness already described goes down first in the form of soft mud in which the voids filled with water are at least 70% of the total volume. Such mud is very soft.

The percentage of voids furnishes, on the whole, the best index of consolidation and stability. In itself, it is not an adequate basis of

comparison, because different kinds of core material may have different degrees of stiffness with the same percentage of voids; but, notwithstanding this difference, the percentage of voids is the best index of consolidation so far available.

The material when it goes down contains 70% of voids. Under these conditions, the minute solid particles are held apart by the water contained between them, and the passages through which water can pass are much larger than they would be with a more compact arrangement of the same material. Water drains from this material gradually, and the material consolidates.

The drainage may be horizontal, in which case the surplus water finds its way through the core material to the toes and escapes. There is, however, a shorter way out, and it appears probable that most of the drainage takes place vertically. That is to say, the solid particles settle down and consolidate while the water moves upward between them to the top. This consolidation takes place gradually at rates depending on the dimensions of the dam and also on variations in the effective size of the core material; for while the writer considers all this material broadly as having an effective size of 0.002 mm., there are no doubt considerable variations in size and permeability.

Experience shows that consolidation goes on so that after a period of from a few months to two or three years in the ordinary course of dam construction, the material will have been consolidated to a point where the voids are about 50 per cent. As the consolidation increases, the sizes of the passages between the grains become smaller, the flow becomes less rapid, and the process of consolidation goes forward more and more slowly.

Material with 50% of voids when taken out of a boring is described as material having the consistency of stiff putty. It is capable of standing up in a boring driven beyond the casing for several feet, and it offers great resistance to penetration.

One of the best field methods of testing the consolidation of core material has been to find the depth to which a 1½-in. pipe could be forced into it by two men. This method of testing has been used in several dams. It has been possible to co-ordinate the penetration so found with records of the percentage of voids. In a general way, penetration by 1½-in. pipe pushed down by two strong men extends to the point where the voids are about 50 per cent. In other words, the 50%

material is so stiff that it can no longer be penetrated to any considerable depth by this method.

Tests of material by letting down cannon balls, etc., are less searching. The penetration does not go as far.

Material with 50% of voids in a considerable depth acts essentially as a liquid. It exerts the full lateral pressure corresponding to its height and weight per cubic foot, and when the resistance of the toe is overcome, this material moves forward and flows. Precise limits cannot be set, but this is the general result of observation at Calaveras as a result of determinations of voids in many samples of material that did flow and in other samples of material that did not flow. This refers to the Calaveras Dam* near San Francisco, Cal., which slipped as it was approaching completion on March 24th, 1918, investigation of which afterward gave an unusual opportunity for studying some of the conditions that grew out of the methods of placing material that had been used. (See Figs. 4 to 6.)

Core material becomes solid when it is dried. Core material dried in the summer sun at Calaveras was consolidated until only 35% of voids remained. In this condition, it was only a little less strong and stable than the natural sandstone in the neighboring hills.

Core material consolidated to 35% of voids would form the strongest and most stable kind of an earth dam. No large toes would be required to contain it, for on any reasonable slope it would be stable by itself. Such consolidation, however, as far as is known, is reached only by complete drying. Unfortunately, drying cannot be applied to the core of a large dam.

Without attempting precision of statement, it is the writer's judgment that if core material like that at Calaveras could be consolidated to 40% of voids, it would be sufficiently stable to do its share toward resisting the pressures that come on the dam without tending to disruption. Perhaps this degree of stability would be reached before consolidation had proceeded as far as 40% of voids. There is a middle ground of uncertainty. Material with 50% of voids is still unstable; and between this limit and 40%, one cannot be certain just where to draw the line.

* See *Engineering News-Record*, April 4th, 1918, p. 679, and December 26th, 1918, p. 1158.

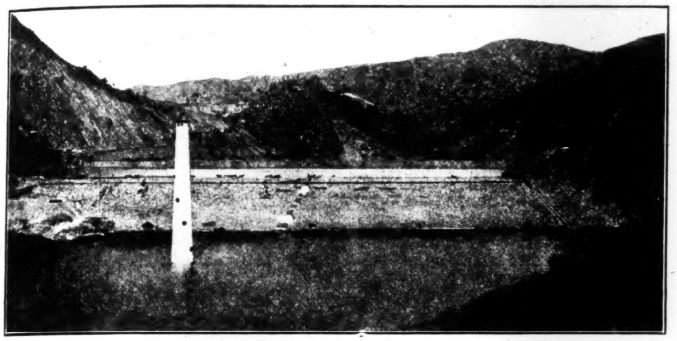


FIG. 4.—CALAVERAS DAM DURING CONSTRUCTION, SHOWING UPPER AND LOWER TOES, WITH POOL BETWEEN. WATER IN FOREGROUND IS IN RESERVOIR ABOVE DAM.

FIG. 5.—CALAVERAS DAM DURING CONSTRUCTION, SHOWING TOES AND CENTRAL POOL A FEW DAYS BEFORE THE SLIP.

FIG. 6.—CALAVERAS DAM AFTER THE SLIP. ON MARCH 24TH, 1918, 800 000 CU. YD. OF MATERIAL MOVED 300 FT. UP STREAM AND DROPPED 100 FT. IN ELEVATION, TURNING ALSO 30° TO THE RIGHT.

It is interesting to note that the 1½-in. pipe used in the penetration tests only reaches to points where the material is still unstable. It follows that tests of this kind do not and cannot demonstrate stable material. Tests made by cannon balls or other less searching means are equally defective. It cannot be considered, in the light of the Calaveras experience, that tests made by such methods throw much light on conditions of stability. They may at times serve a purpose in showing lack of stability, but that is as far as use can be made of them.

It is the writer's thought that the best method of investigating consolidation and stability is by making borings. There are difficulties, for boring usually involves stopping the sluicing of material while the holes are being put down; but it does lead to more definite results. Borings can be driven and samples taken from the bottom of the borings for physical inspection and for determination of the percentage of voids.

The Determination of the Percentage of Voids.

This is a very simple matter, but experience shows that it is easy to go wrong, and for that reason a brief statement of methods that have been found suitable will be given.

The samples taken from the bottom of test borings are saturated with water. A determination of water content, therefore, can be made a measure of voids. This procedure is sound so far and only so far as the voids are completely filled with water. A useful check can be obtained from the specific gravity of the moist material.

The specific gravity of the solid particles of core material may be taken as 2.65. It may be that in some locations other specific gravities will be found, but, in the writer's experience, actual values differ little from this mean value.

In determining the specific gravity of these exceedingly small particles, the most careful manipulation is required to get all the air out of the dry material. Methods similar to those used for determining the specific gravity of Portland cement are appropriate, but water can be used.

A table can be made showing the weight of the solid particles, the weight of water, the percentage of water by weight, and the specific gravity of core material containing various percentages of voids.

Table 1, in convenient and somewhat condensed form, gives these values for saturated clay.

TABLE 1.—SATURATED CLAY.*

Specific Gravity of Solid Particles, 2.65.

Percentage of voids.	Percentage of water, by weight.	Specific gravity. Wet.	WEIGHT PER CUBIC FOOT.	
			Wet.	Dry.
40	20.1	1.990	124.2	99.2
41	20.8	1.974	123.1	97.5
42	21.5	1.957	122.1	95.9
43	22.2	1.940	121.1	94.2
44	22.9	1.924	120.0	92.6
45	23.6	1.908	119.0	91.0
46	24.3	1.891	118.0	89.3
47	25.0	1.874	117.0	87.6
48	25.8	1.858	116.0	86.0
49	26.6	1.841	115.0	84.3
50	27.4	1.825	114.0	82.7
51	28.2	1.808	112.9	81.0
52	29.0	1.791	111.9	79.3
53	29.8	1.775	110.8	77.7
54	30.6	1.759	109.8	76.0
55	31.5	1.743	108.7	74.4
56	32.4	1.726	107.6	72.8
57	33.3	1.709	106.6	71.1
58	34.2	1.692	105.6	69.5
59	35.1	1.676	104.5	67.8
60	36.1	1.660	103.5	66.1
61	37.1	1.643	102.5	64.5
62	38.1	1.626	101.5	62.8
63	39.1	1.609	100.5	61.2
64	40.1	1.593	99.5	59.5
65	41.2	1.577	98.5	57.8
66	42.3	1.560	97.4	56.2
67	43.4	1.543	96.4	53.5
68	44.5	1.527	95.4	52.9
69	45.6	1.511	94.4	51.2
70	46.8	1.495	93.4	49.6

* This table is not to be used for any material that has lost water by evaporation so that the voids are not completely filled with water.

When a sample is taken from a test boring, a portion is put in a dish and weighed. The weighed material is dried at or above the boiling point to constant weight, and the percentage of moisture computed. As even a little drying before the first weight affects the result, the samples should be weighed on the spot when first brought to the surface; to do this, portable equipment must be used. After the first weighing, samples may be taken to the office or laboratory for the completion of the process. When the percentage of moisture

by weight is found, the corresponding percentage of voids is obtained from Table 1.

As a check, the specific gravity of another part of the sample is determined. This is obtained by putting a weighed portion of the moist mass into a graduated cylinder partly filled with water and noting the increase in water level. The specific gravity is the ratio between the weight of material added and the weight of the water that corresponds to the increase in level, and so to the volume of material placed in the cylinder. With this specific gravity determined, Table 1 indicates the percentage of voids.

The percentage of voids determined by the two methods with good manipulation should check within 1 or 2 per cent. If it does not, there is something wrong and the error must be found and eliminated.

To determine the weight of the coarse and hard toe material, no better method has been found than to make an excavation accurately to a dimension of 1 cu. yd., or other convenient size, and to weigh the excavated material on platform scales. Percentages of moisture vary, and the weight of dry material affords the best basis of comparison. An average sample of material on the scales is taken for determining the percentage of moisture, and the results are reduced to weight of dry material per cubic foot.

Draining Fine Material.

The writer is fully satisfied that there is no practical way of rapidly draining core material as fine in grain size as that described previously. He has considered it from a theoretical standpoint and also from the standpoint of practical experience.

After the material reaches a condition of 50% of voids, 6 cu. yd. must be compressed into 5 cu. yd. of 40% material by the exclusion of 1 cu. yd. of water before full stability can be counted on. This water, amounting to one-sixth of the whole volume of core material, must be expelled. The computed rate of flow through material with an effective size of 0.002 mm. is very slow. The rate depends on, and is proportional to, the hydraulic slope. With a slope of 10% (which is as steep as is usually found) the computed rate of flow as a solid column of the full area of the section is 0.32 mm. daily. At this rate, 80 days are required for the water to move 1 in. and 2.6 years for it to move 1 ft. This calculation cannot be used with assurance as estab-

lishing the exact rate of flow, but it is the writer's judgment that the actual rate is as likely to be below as above the result given. It is clear that, for a large dam, complete drainage and consolidation may be a matter of many years.

From a practical standpoint, it may be recorded that a 6-in. well driven 60 ft. in saturated core material having voids averaging 50%, with the casing perforated freely to admit water, stood empty for weeks. Not enough water drained from this material to raise the water in the well. This may represent the effect of capillarity. With such a degree of tightness, it is clearly useless to attempt to secure consolidation by any method depending on additional drainage outlets.

Measuring Core Pressures.

An interesting method of measuring the pressures actually existing in core materials has been suggested by A. T. Goldbeck, Assoc. M. Am. Soc. C. E.* By this method, small test cells are built into the core material. Some are placed with horizontal and some with vertical faces, arranged to indicate pressures on the faces during construction and afterward. Such devices are reported to have been placed in hydraulic-fill dams now building by the Miami Conservancy District. The results of observation through a certain period have already been recorded. These observations indicate, as would be expected, that the horizontal and vertical pressures are the same at first and until the material has become somewhat consolidated. Afterward, horizontal pressure increases less rapidly than vertical pressure, and the inference may be drawn that this represents solidification of the material in which the cells are placed to a point where the full horizontal pressure is no longer produced.

This is certainly an interesting method of test and one that may throw light on the conditions of core material.

It reminds the writer of one of his early experiences, which related to different circumstances, but possibly the same underlying principles are involved. It was proposed to build large and heavy structures in which the weight would be well distributed on a foundation of stiff silt. The question was presented as to whether the foundation was sufficient to carry the weight. An apparatus was provided by which test areas of 1 sq. ft. at the level of the proposed foundation were

* *Engineering News-Record*, April 18th, 1918, p. 758.

loaded. The results of the tests were most satisfactory. The material carried more than the expected loads, with only insignificant settlements. When the actual structure was built, however, there was considerable settlement. The actual settlement was many times greater than had been indicated by the tests.

The explanation is simple enough. The single square foot that was loaded distributed the weight applied to it to a much larger area a short distance below. What was loaded was in effect a pyramid with a flat top of 1 sq. ft. The full applied weight per square foot was carried only at the top, and the unit stress rapidly became less going downward as the area of the pyramid increased. A thin layer of material just below the footing actually got the pressure and was compressed accordingly, and in this way the slight observed settlement was produced. Lower down, however, the unit pressure was lower and there was no appreciable compression.

When the structure was built there was no chance for a corresponding distribution of pressure. The whole area underneath was loaded, and the weight had to be carried through the full depth of silt and the silt was compressed. A corresponding settlement would no doubt have taken place in the test apparatus if the material loaded had been a column 1 ft. square all the way down to rock. The writer thinks that because of this dispersion of load it is generally true that a small test area will carry more per unit than a larger one, and it may be that the same conditions will be found to apply to the test cells built into the core of a dam.

In view of all the information now available, the only safe course to follow, so long as the core material is like that here described, is to assume that it will produce the full horizontal pressure corresponding to its weight and height; and the toes must be made large enough to resist that pressure with ample safety.

Resisting the Pressure of the Core Material.

The first point to be considered is the coefficient of friction of the material in the toe, sliding either upon the foundation or upon itself. With hard, clean gravel a coefficient of 0.7 or 0.8 might reasonably be expected, although the data for establishing this coefficient are not as convincing as could be desired. No such coefficient was found, however, in accounting for the Calaveras slip. Instead, it was esti-

mated that the weight of the material pushed forward in the up-stream toe was five times as great as the pressure of core material against it. In estimating that pressure the whole height and weight of core material was included, assuming that it acted as a heavy liquid. This indicated a coefficient of friction of only 0.20. This is to be taken as an average for the whole area on which the slip occurred. It may be recorded, however, that a similar calculation for the lower toe which did not slip indicated a coefficient of 0.22. No one can tell how much more it would have held.

The Calaveras material is softer and perhaps more slippery than the hard particles of glacial drift of the Eastern States; and it would not be reasonable to expect as high a coefficient of friction. It may be, also, that the coefficient of friction under such heavy pressures as are found in the lower part of a high dam will be less than under the small pressures of moderate depths.

Some experiments were authorized by George Wilhelm, M. Am. Soc. C. E., and made by G. W. Hawley, M. Am. Soc. C. E., Resident Engineer at the San Pablo Dam, near Berkeley, Cal. (see Fig. 7), following suggestions of the writer. A 14-in., cast-iron pipe was cut into two short parts, one of which was fixed and the other attached securely to a steel frame which swung freely on an axis about 8 ft. above. Another steel frame securely connected the ends of this axis with the fixed part of the 14-in. pipe and with other parts of the apparatus. Two hydraulic jacks were used and the pressures were computed from the measured pressures on the pistons.

The two pieces of 14-in. pipe were placed in line and the frames held them so that the ends would just clear.

A sample of the material to be tested was placed in the pipe, filling the lower or fixed part and extending some distance into the movable part. Oak planks, cut to fit as a loose piston, were then placed above. One of the hydraulic jacks was placed above it. Pressure was then applied to compress the material in the pipe to any desired extent. Experiments were made with various pressures, the greatest corresponding to a depth of fill of 200 ft. While the material was held under this pressure, the other jack was placed horizontally against the side of the 14-in. pipe with suitable blocking, and pressure applied until there was movement. The ratio of pressures on the two jacks then gave the coefficient of friction.

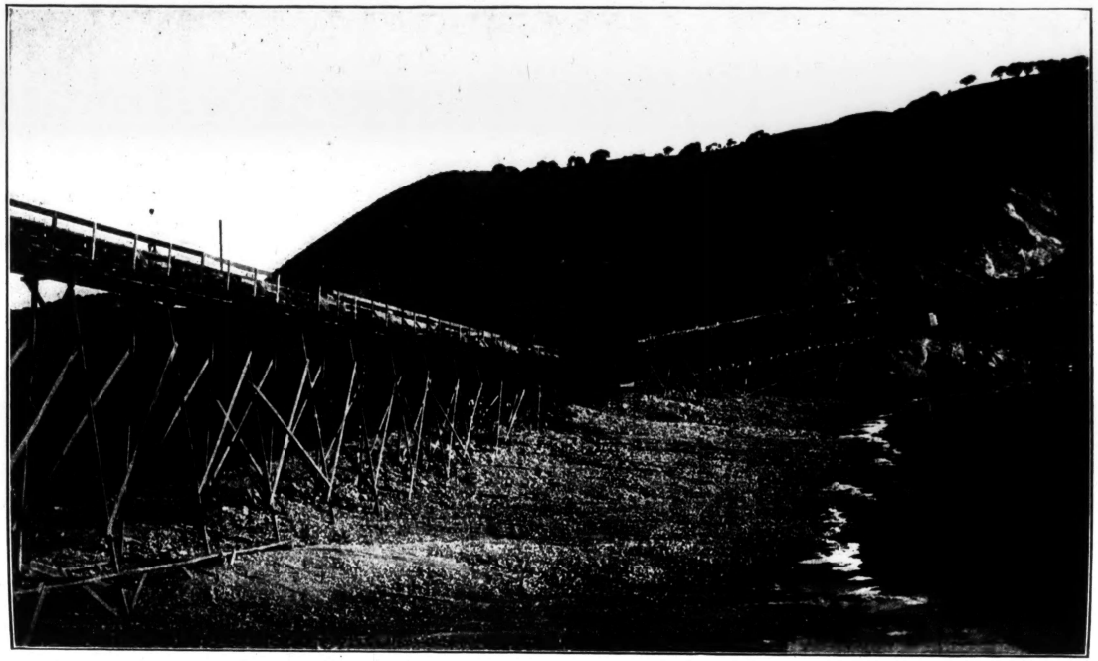


FIG. 7.—SAN PABLO DAM, SHOWING DEVELOPMENT OF LOWER TOE, WITH THE FINER MATERIAL PASSING OVER IT TO THE CENTRAL POOL.

The steel frame and the pivot at the top were built so that there was no appreciable friction in the apparatus. A slight correction was necessary because the lever arm to the slipping plane was longer than the lever arm to the point on the 14-in. pipe where pressure was applied.

The form of the experimental apparatus was suggested to the writer by a paper by Mr. A. L. Bell on clay pressures.* The apparatus as worked out was more like that used by E. P. Goodrich, M. Am. Soc. C. E., in his experiments on soil pressures.†

The apparatus used differed from both of its predecessors in that tests were made at very much higher pressures. The results may be compared, but it must be remembered that the local material had peculiarities of its own, and that it was no doubt quite different physically from the material tested by Messrs. Bell and Goodrich.

The writer will not attempt to discuss this matter further, but suggests it as a fruitful topic for further experiment.

Experiments were made by Mr. Hawley with various local materials at various pressures. These showed that, with low pressures, the coefficient of friction was distinctly larger than it was with higher pressures. The explanation for this may be that the higher pressures broke down and crushed the particles, and so reduced the relative resistance. After a certain pressure was reached there was less rapid change in the coefficients, and for the higher pressures the coefficients for any material were practically constant.

For material from the local soft sandstone rocks at San Pablo, coefficients of friction of approximately 0.5 were found.

There was one kind of material more slippery than the others for which an average coefficient of 0.45 was found. Other harder materials gave coefficients of 0.55 and more. Sand and gravel from the creek representing the remnants of much harder rocks in the hills (and so corresponding more nearly to the glacial drift of the Eastern States) gave coefficients of about 0.7.

The materials at San Pablo are similar to the materials at Calaveras; and there is reason to think that the coefficients found at San Pablo would apply approximately to the Calaveras materials.

* *Minutes of Proceedings*, Inst. C. E., Vol. CXCIX (1914–15), p. 233.
† *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 297.

What then is the explanation of the fact that the Calaveras Dam slip indicated a coefficient of 0.2 while similar materials in the test apparatus show a coefficient of 0.5—two and one-half times as much?

The answer seems to lie in the fact that in the course of construction at Calaveras, before the dam was very high, the construction pool in the center of the dam was sometimes quite wide; so that at some levels the greater part of the width of the dam was composed of fine-grained core material. Afterward the proportion of solid fill was increased and the width of core reduced, but the effects of the wide pool may have been permanently left in layers of core material extending far out under the more solid parts of the toes.

It seems probable that unstable core material placed in this way furnished the lubricant that facilitated and made possible the slip in the dam.

Section of Dam Required with Core Material Considered as a Fluid.

At Calaveras, the toe material was slightly heavier per cubic foot than the core material. At San Pablo, with fragments of porous sandstone rock predominating in the toes, there was scarcely any difference. With harder material like the glacial drift of the Eastern States, the toe material would be considerably heavier.

For the purpose of a first rough calculation, assume that the weights per cubic foot of toe and core material are equal, as they are at San Pablo. If a coefficient of friction of 0.5 is assumed, each toe must weigh twice as much as the pressure exerted by the core. The pressure exerted by the core is $\frac{1}{2} wh^2$ and the weight of the toe to balance it must be wh^2. To produce this weight, a uniform outer slope of 2 to 1 will suffice. In addition, a top width equal to the maximum width of core must be used. If a factor of safety of 2 is required, the outside slope must be made 4 to 1, but in view of the records of slips in actual dams, it may well take a higher factor of safety. A slope of 5 to 1 will give a factor of safety of 2.5 and a slope of 6 to 1 will give a factor of 3. Where toe material is considerably heavier than the core, as would be the case with harder and heavier rocks, the slopes could be reduced in proportion to the increased specific gravity without reducing the factor of safety. Thus, with toe material 20% heavier than core material, a slope of 1 to 5 would allow a factor of safety of 3. Very few hydraulic dams have been built with sections as large as this

line of thought suggests. The Gatun Dam at Panama heads the list, and is even larger.

A great many cubic yards are involved in such a large section, but if the work is done cheaply enough per cubic yard, it may be economical to use the extra volume. Safety can be secured in this way; and it may be that this is the best procedure to follow.

Dimensions of a Number of Dams.

To see what these slopes would call for in the way of section in comparison with earth dams that have been actually built either by the hydraulic method or as dry fill, the following comparison is made. Assuming a freeboard of one-tenth the greatest depth of water, H, and a top width of 0.25 H and uniform slopes, the cross-sections of dams in terms of the greatest depth of water would be as follows:

Slope.	Volume.
3 to 1	3.9 H^2
4 to 1	5.1 H^2
5 to 1	6.3 H^2
6 to 1	7.5 H^2

The actual dimensions of a number of dams are given in Table 2.

Rock-fill dams are not included, but rock fill forms some part of a number of dams in the list. No dams holding less than 64 ft. of water are included.

The Necaxa Dam slipped, but was afterward rebuilt on the original section. The Big Meadows Dam is being raised with an enlarged section.

Increasing the Grain Size of Core Material.

The second way of increasing stability is to use core material that will consolidate more rapidly and thoroughly. There is only one way to insure this; that is to increase the grain size of the material allowed to remain in the core. With pools and water quantities actually used in the dams known to the writer, particles smaller than 0.001 mm. and perhaps up to 0.002 mm. have been wasted. Everything larger has gone into the dam.

In some cases, an effort has been made to secure completeness of deposition of these fine particles to avoid waste and to increase the yardage accounted for in the dam.

TABLE 2.—DIMENSIONS OF A NUMBER OF EARTH DAMS THAT HAVE STOOD SUCCESSFULLY.

Dam.	Height to flow line, H.	Area of section, in square feet, A.	Ratio, $\frac{A}{H^2}$	Freeboard.	Width, in Feet.				Bottom, W.	Ratio, $\frac{W}{H}$
					Top.	At flow line.	50 ft. below flow line.	100 ft. below flow line.		
*Gatun	78	99 200	16.30	30	100	397	1 469	....	1 990	25.5
*Big Meadows	64	24 300	5.93	20	30	160	460	....	548	8.56
*Coquitlam	83	38 700	5.61	15	40	145	521	....	702	8.48
*Cambria	82	31 240	4.65	13	20	98	429	....	685	8.34
Ashokan	90	35 400	4.37	20	34	112	400	....	650	7.23
Druid Lake	82	27 900	4.15	5	60	90	390	....	582	7.10
San Andreas	80	26 200	4.10	5	30	58	387	....	606	7.58
*Paddy Creek	120	58 200	4.05	25	23	161	414	664	757	6.30
*Somerset	92	33 200	3.94	18	25	78	375	....	616	6.67
San Leandro	65	16 350	3.88	5	30	56	335	....	457	7.04
*Haiwee	75	21 600	3.85	14	20	90	340	....	465	6.20
Croton	96	33 400	3.62	24	30	126	336	....	528	5.50
Tabeaud	86	26 400	3.57	8	20	60	345	....	553	6.44
*Necaxa	164	90 800	3.36	16	54	136	3-6	636	954	5.81
Cold Spring	88.5	26 200	3.34	10	20	70	320	....	512	5.79
Belle Fourche	100	33 100	3.31	15	20	65	284	656	656	6.56
Lahontan	112	40 900	3.25	12	20	80	330	580	640	5.72
Santa Maria	85	23 500	3.25	8	20	60	310	....	485	5.71
Pilarcitos	74	17 600	3.22	5	30	56	288	....	415	5 61
Morris	90	25 700	3.18	9	20	60	309	....	502	5.58
Borden Brook	64	12 700	3.10	7	24	49	280	....	397	5.27
Honey Lake	90	25 000	3.08	6	20	50	300	....	590	5.56
Goose Creek	137.5	54 900	2.90	7	16	54	304	554	741	5.39

* Hydraulic fill, wholly or in large part.

To increase the effective size of core material, it is necessary to increase the limit of size of particles to be wasted. This can be done by narrowing the pool or by increasing the quantity of water passing it (see Figs. 3, 8, 9, and 10). On one occasion at San Pablo, to test this out, the toes were allowed to increase in height for a time without raising the pool. This was continued until the pool almost disappeared. Hardly more than a muddy creek remained in the middle of the dam. The core became comparatively narrow and much coarser in grain size. The effective size was increased to approximately 0.01 mm. Finer particles were all wasted. In other words, the core material was, say, five times as great in grain size. It had, therefore, twenty-five times as great draining capacity, and only one twenty-fifth of the capacity to hold water by capillarity.

Material placed in this way had a much better chance of drainage; first, because the core was narrower and there was less water to be

FIG. 8.—SAN PABLO DAM, SHOWING MAXIMUM WIDTH OF CENTRAL POOL.

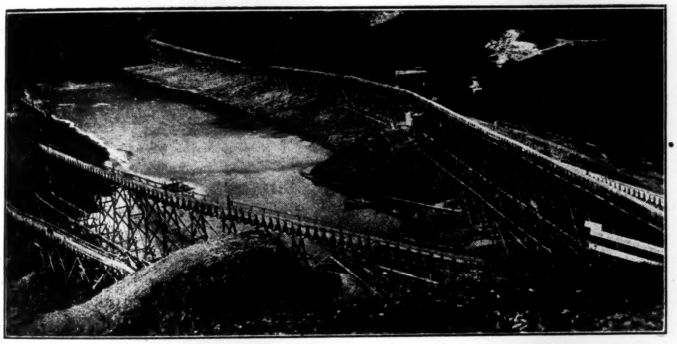


FIG. 9.—SAN PABLO DAM, SHOWING ORDINARY WIDTH OF CENTRAL POOL PRODUCING CORE MATERIAL 0.002 MM. IN SIZE.

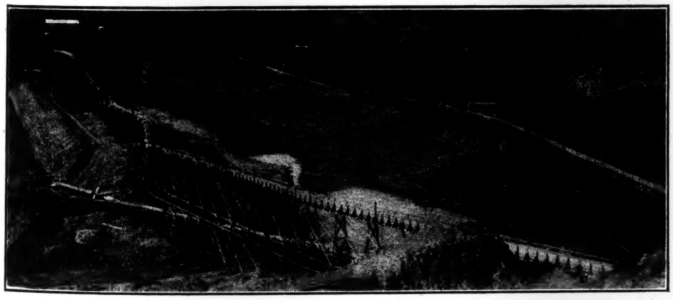


FIG. 10.—SAN PABLO DAM, SHOWING CENTRAL POOL GREATLY REDUCED IN WIDTH FOR PURPOSE OF INCREASING GRAIN SIZE OF CORE MATERIAL. IT WAS PUSHED TO A NARROWER WIDTH THAN SHOWN.

removed; second, because the water had a shorter distance to go; and third, because, other things being equal, the water would get away twenty-five times as fast. After a few weeks of operation in this manner, the work was stopped temporarily, which gave an opportunity to examine the deposited core material. After only a few days it became hard enough to walk on, many times more solid than ordinary core material.

In one other respect this coarse-grained puddle differed from ordinary puddle. There was a small relative range in its grain sizes. The fine particles were eliminated, and the volume of puddle produced was reduced, but there was no very great change in the quantity and character of the coarser particles. These coarser particles constituted the whole mass of the coarse-grained puddle. With ordinary puddle, they were mixed with a large additional volume of fine-grained stock. The grains in the coarse-grained puddle were thus more nearly of the same size, and it followed that, other things being equal, the percentage of voids was greater. This was found to be the case in the test at San Pablo, and after the material had become hard and stable, it had a percentage of voids that would have indicated lack of stability in ordinary stock. This is an illustration of the fact that voids used as an index of stability can only be properly compared for the same kind of stock.

If this method of increasing the grain size of core material was to be followed throughout the construction of a dam, obviously there would be two results. First, a great deal of fine material would be wasted. The quantity would depend on the quantity of such material in the stock used. In the natural course of events most of this would be deposited harmlessly on the bottom of the reservoir. Second, a dam built in this way would be much more stable but theoretically not quite water-tight. There would be an appreciable amount of seepage through core material of this grain size.

The amount of seepage loss may be estimated approximately. Assuming that the laws of flow are the same as they are in the finest sand to which experiments have extended, it was estimated that at the San Pablo Dam if the whole core material could be made 0.01 mm. and if the remainder of the dam had contributed nothing to tightness, the theoretical rate of seepage would be 280 000 gal. per day. The loss of this quantity of water would not be a serious drain on the supply,

if it adds to stability. Practically, as is known from filter experience, considering the core material as a filter, it would soon silt up and become water-tight.

As a practical proposition in dam construction, core material having a grain size of 0.01 mm. is to be accepted as sufficiently fine from the standpoint of water-tightness. This size of 0.01 mm. is not given as a precise limit. It may be that still coarser material would give sufficient tightness or that finer material would drain sufficiently. The size is mentioned because it is the size of material that was actually found possible to produce at the San Pablo Dam by reducing the dimensions of the pool to a minimum, and because other considerations suggest that it may be suitable.

The question remains as to whether a dam in which the pool and water quantities were adjusted so as to hold the effective size of core material at 0.01 mm., or at some other selected limit, and in which the core material was never permitted to become very wide, would consolidate itself as the dam increased in height to an extent that would eliminate lateral pressure of core material.

It is the writer's idea, that a dam built in this way would certainly be more stable and safer than one in which the core material contained an additional quantity of finer particles.

This thought is similar to an idea expressed by D. C. Henny, M. Am. Soc. C. E., who stated:*

"Probably the nearest approach to a perfect core of great thickness which can be hydraulicked, is one composed of fine sandy silt, such as is generally found in the arid West, having little cohesiveness, good self-drainage qualities, becoming hard and solid after a short time, and yet being, if not perfectly, at least practically water-tight."

Mr. Henny's description may be taken as an accurate description of the writer's idea of 0.01-mm. material. It certainly does not apply to 0.002-mm. material. Mr. Henny's other propositions in regard to fine core material such as is ordinarily used, are believed to be well founded.

The Use of Rock Fill.

In building hydraulic-fill dams, the débris above the solid rock has been most easily worked and extensively used. Such débris is frequently fine in grain size, and its use may result in an excess of core

* *Transactions*, Am. Soc. C. E., Vol. LXXIV (1911), p. 82.

material and a deficiency of toe material. To correct this tendency, rock fill has been added to the toes of a number of dams.

The rock fill has usually been placed as dry fill; that is to say, by blasting, steam shovel, and cars or carts. The cars or carts run on the toes of the dam and deposit material at the same time that the hydraulic process is being used to fill the interior.

At the San Pablo Dam (see Figs. 11 and 12), a similar result has been reached by handling the rock after blasting by the hydraulic method. Open flumes were used with a flow of from 15 to 20 cu. ft. per sec. A 6% slope was necessary as long as wooden flume bottoms were used. With steel flumes, equally favorable results were obtained with a 4% slope. About 3 000 cu. yd. per day were placed, on an average. Pieces of broken rock up to 1 cu. ft. in size were handled under these conditions. By blasting the hardest rock available, it was possible to get material for fill that contained relatively few fine particles, and all of the small quantity of very fine material was wasted. Large additions to the toes of the dam were made in this way.

Working a quarry with a high face, it was possible to throw down great quantities of material in one large blast, and to wash many thousands of yards into the dam from a single position of monitors and flumes. The economy resulting from operating so long in one place, as compared with moving the equipment all over the hillside to pick up scattered and relatively thin deposits of débris above the rock, turned out to be fully equal to the additional cost of rock excavation; and rock fills, built in this way, cost no more per cubic yard than fill made from débris.

On the other hand, the fill made in this way is believed to be heavier, and to have a higher coefficient of friction which gives it additional value as toe material. The practical methods of handling rock fill were worked out by Mr. Hawley and D. W. Albert, Assoc. M. Am. Soc. C. E.

Quicksand Conditions in Dams.

When a granular material has its pores completely filled with water and is under pressure, two conditions may be recognized. In the first or normal case, the whole of the pressure is communicated through the material from particle to particle by the bearings of the edges and points of the particles on each other. The water in the pores is under no pressure that interferes with this bearing. Under such conditions,

the frictional resistance of the material against sliding on itself may be assumed to be the same, or nearly the same, as it would be if the pores were not filled with water. In the second case, the water in the pores of the material is under pressure. The pressure of the water on the particles tends to hold them apart; and part of the pressure is transmitted through the water. To whatever extent this happens, the pressure transmitted by the edges and points of the particles is reduced. As water pressure is increased, the pressure on the edges is reduced and the friction resistance of the material becomes less. If the pressure of the water in the pores is great enough to carry all the load, it will have the effect of holding the particles apart and of producing a condition that is practically equivalent to that of quicksand.

An extra pressure in the water in the pores of such a material may be produced by a sudden blow or shock which tends to compress the solid material by crushing the edges and points where they bear, or by causing a re-arrangement of particles with smaller voids. An illustration of this can be seen in the sand on the seashore. Such sand, comparable to dune sand in size, is usually found to be saturated with water for a certain distance above the water level. This condition is maintained by capillarity. If a weight is slowly placed on this saturated sand, there is a slight settlement, the grains of sand coming to firmer bearings, and the weight is carried. A sharp blow, as with the foot, however, liquefies a certain volume and makes quicksand. The condition of quicksand lasts for only a few seconds until the surplus water can find its way out. When this happens the grains again come to solid bearings and stability is restored. During a few seconds after the sand is struck, however, it is almost liquid, and is capable of moving or flowing or of transmitting pressure in the same measure as a liquid.

Fine-grained sand in which this condition exists is called quicksand. The properties of quicksand are well known. Fine-grained sand saturated with water and then mixed with an additional 5% of water acts practically as a liquid. It will flow through small orifices or in a pipe at a 5% slope. The sand in a mechanical filter in process of washing is a typical case. When the sand is drained, in the filter, it forms a bed as hard as any sand bed. When subjected to a reverse current of water strong enough to lift it slightly, however, the volume is increased by perhaps 5%, and it becomes liquid. An object can

FIG. 11—EXCAVATING ROCK AT SAN PABLO DAM BY HYDRAULIC JET, AFTER BLASTING.

FIG. 12.—PLACING ROCK HYDRAULICALLY IN SAN PABLO DAM, FRAGMENTS UP TO 1 CU. FT. BEING HANDLED BY THIS METHOD.

then be pushed through it with scarcely more resistance than would be offered by a liquid of high specific gravity.

The conditions that control the stability or lack of stability in quicksand may also control the stability or lack of stability of materials in dams.

The puddle clay core of an hydraulic-fill dam is physically like quicksand, but with particles one hundred times smaller in diameter and a million times smaller in weight. It has the instability of quicksand in full measure and it retains it for a long time, or perhaps indefinitely.

With the coarse-grained part of an hydraulic-fill dam, that is to say, with rock toes, there may be also, at times, a similar condition. Generally speaking, it would be expected that such coarse-grained materials would have sufficient drainage to let out surplus water and prevent the possibility of an excess sufficient to destroy its stability. With hard-grained materials from glacial drift of New England and the Northern States, it is hard to conceive of a lack of drainage in gravel that would permit the accumulation of an excess of water. With the softer materials of the Pacific Coast, however, the conditions may be different. In the first place, these soft rocks by partial crushing under pressure produce fine material which tends to fill the remaining spaces and to reduce the drainage capacity.

On the other hand, each increment of loading applied to soft-grained material produces a certain compression and settlement; and with it a reduction in voids. This may happen to a toe of soft rock on the up-stream side of a dam against which water is being stored during construction. There is first an open condition with ample voids and ample drainage. As the dam is built higher, pressure increases; there is compression and reduction in porosity. Each additional increment of loading and compression means that a certain quantity of water representing the difference between the old voids and the new voids must be expelled. As long as the material remains sufficiently pervious to carry off this excluded water, the process of compression is harmless. The surplus water is pushed back into the reservoir and stability is retained. There may come a time, however, when the compression goes forward so rapidly that the surplus water cannot be carried off fast enough. When that point is reached (if it is reached), there will tend to be an excess of water in the interstices

and that excess will transmit some of the pressure that was before carried only by the bearings of solid particles, and the frictional resistance of the material will be less, and perhaps much less, than it was before.

The thought has occurred to the writer, in looking at the material that slid in the Calaveras Dam, that something of this kind may have happened on a large scale—800 000 cu. yd. of fill flowed for a brief space, and then became solid. It was, in fact, so solid that in examining it afterward, by samples and by borings, it was difficult to see how the material could have flowed—as it certainly did flow.

It may be that after the first movement there was some readjustment of the material in the toe which resulted in producing temporarily this condition of quicksand, and which destroyed for a moment the stability of the material and facilitated the movement that took place.

This will not account for the initial movement; but the initial movement of some part of the material might result in accumulating pressure, first on one point, and then on another, successively, as the early points of concentration were liquefied and in that way a condition comparable to quicksand in a large mass of material may have been produced.

Summary.

To summarize briefly the points that the writer has attempted to make and to apply them practically to dam construction:

1.—It is not well to build an hydraulic-fill dam of material of which any large percentage consists of clay or of particles less than 0.01 mm. in diameter; and in general all such smaller particles may well be wasted and excluded from the dam.

2.—By reducing the construction pool to a minimum, and by controlling it and the quantities of water used for sluicing, the core material may be held to a certain degree of coarseness by wasting all smaller particles. An effective size of 0.01 mm. may reasonably be sought.

3.—To study by borings the actual consolidation of the material, and to adjust the construction of the upper parts of the dam to the demonstrated condition of that which lies below.

4.—To make the toes large enough to resist with an ample factor of safety the whole pressure of the core material as a liquid until there

is demonstration of the solidification of the core to a point where horizontal pressure is eliminated.

5.—To increase the weight and solidity of toes by the use of rock fill, placed hydraulically or otherwise.

6.—Stability is increased by compactness. It is worth while to watch voids closely, and to make every effort to hold them at a minimum. The extra weight is advantageous, but security against compression and re-arrangement with resulting temporary quicksand conditions, can be best reached in this way.

A strict application of these principles may reduce the number of hydraulic-fill dams that are built by eliminating that method of construction from consideration where the available material contains too many very small particles. It may also increase considerably the volume and cost of those that are built. It would seem, however, that following them to a logical conclusion, with such testing as can reasonably be done, will result in eliminating present uncertainties and in putting a most useful method of dam construction on a definite and safe basis.

DISCUSSION

Mr. Curtis.

C. E. CURTIS,* M. AM. SOC. C. E. (by letter).—This interesting and instructive paper dwells on two important elements in hydraulic-fill dam construction, core material and toe material, discussed with greater care and in greater detail than the writer has yet seen. The summary is valuable because it states, briefly and clearly, important facts well worth consideration by any one engaged in building hydraulic-fill dams. It makes available, in a few sentences, facts that otherwise might require much reading and investigation to obtain. The important points are brought to the reader's attention and the things necessary to bear in mind for a successful dam built by the sluicing method are indicated.

Two important features to be carefully considered in every hydraulic-fill dam are the fine, plastic, primarily impervious, unstable core and the coarse, stable, pervious, supporting outer embankment, spoken of by the author as the toe. It is essential to have a water-tight core and equally essential to provide sufficient coarse material to hold the core in place and give stability. The whole secret of hydraulic-fill dam construction, in the writer's opinion, is the proper handling of these two features without making either excessive and, at the same time, obtaining the desired result, namely, a tight and safe structure. The details of handling these features are, of course, necessarily different in every operation, because of the varying conditions, but the principle is the same always.

The writer was much interested in the description of the pipe tests used to determine the solidity and character of core material, because of the similarity in both methods and results to those obtained in his own experience. He found that 30 or 35 ft. was about the limit that two men could push a 1-in. pipe down into the soft material of the core, and that 60 ft. was about the limit the same pipe could be driven by using hand hammers. Samples taken from the pipe were quite uniform and the most satisfactory of any obtained. There was always the feeling, however, that a sample obtained in this way was more compact, due to the manner of taking it, than the material really was in the dam, but samples so taken are the best that can be obtained, as far as the writer's experience goes.

The author states that he found it necessary to stop sluicing in order to make pipe tests. The writer has known of instances where such tests were usually made while sluicing was in full progress, without any trouble or inconvenience, the work being done by men on a raft.

In a locality where fine material predominates and it is difficult to get coarse material in large quantities, it might become necessary

* Ithaca, N. Y.

to waste the very fine particles of the core material in order to secure compactness, but, on the other hand, in a locality where coarse material predominates, as it does in some places, it is perhaps not always necessary to do this, but the finer the core material the more necessary and difficult it becomes to hold the fine particles where they belong. It can be done successfully only by providing an adequate mass of coarse material well and carefully placed. Mr. Curtis.

Some years ago the writer was in charge of an hydraulic-fill dam where great care was exercised to retain all the fine material possible in the core, as it was thought to be necessary for tightness and would also add to the volume in the embankment. This dam, more than 90 ft. high, was built without any slips or constructional mishaps and is unusually tight with little if any seepage through it. There was, however, an abundance of both coarse and fine materials advantageously located for sluicing so that the proportions could be easily controlled. The work referred to is the Quemahoning Dam of the Cambria Steel Company, near Johnstown, Pa.

Fig. 13 shows a cross-section of the completed dam with the general sub-divisions or zones of material indicated approximately as the dam was built. The late Frederic P. Stearns, Past-President, Am. Soc. C. E., was the Consulting Engineer for the dam; Joseph Morgan, Consulting Engineer for the Cambria Steel Company, was Consultant on the dam and the pipe line; and John W. Gocher, Chief Engineer of the Cambria Steel Company, was Chief Engineer of both. The writer had charge of the construction of the whole development including both the dam and the pipe line. Fig. 13 contains several brief statements of facts pertaining to the dam, and the following notes in regard to the pipe line, which was built at the same time and is a part of the development, are pertinent.

The distance from the dam to the works by pipe line is 14 miles; the pipe is 5 ft. 6 in. in diameter and its total weight is 13 000 tons; it crosses Stonycreek River thirteen times, passes through five tunnels aggregating 9 000 ft. in length and the daily flow is from 75 000 000 to 90 000 000 gal. The catchment area is 92 sq. miles and the flooded area 900 acres.

This dam, 93 ft. high above the natural surface of the valley, was built with flat slopes of 4 to 1 for the lower and 3 to 1 for the upper half of the embankment; both the up-stream and down-stream slopes were the same. The width of the spillway is 250 ft., the maximum calculated flow over which is 11 800 000 000 gal. per 24 hours. The dam is 950 ft. long on top and contains about 600 000 cu. yd. of material, of which more than 500 000 cu. yd. were sluiced. The two dikes (marked *d*) one at each toe, were built by team and scraper of material taken from the middle third of the area covered by the dam. A certain limited quantity of material on the top was placed by hand

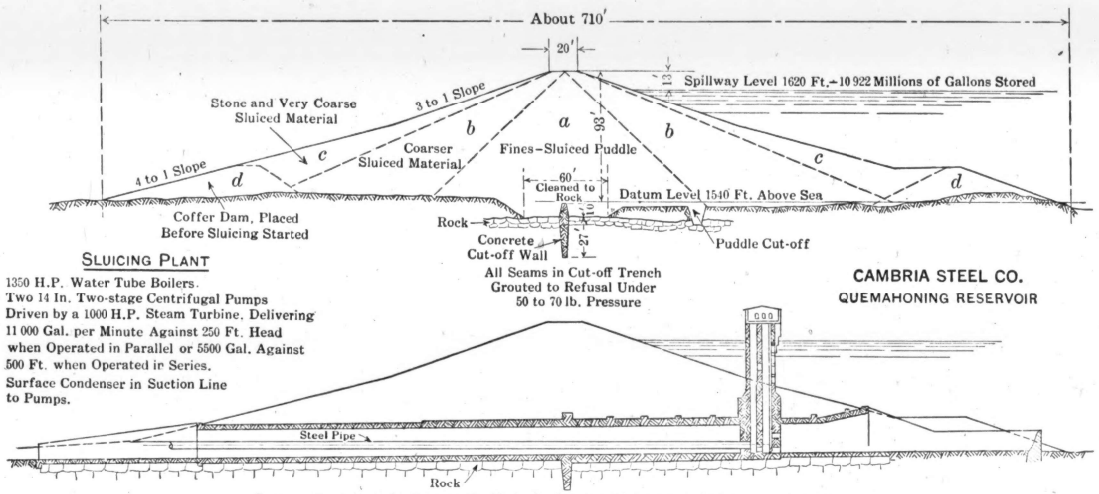
Mr. Curtis.

in the usual manner, all other material having been sluiced. The sides of the valley were about 200 ft. higher than the top of the finished dam, so there was sufficient elevation to secure the desired grade for sluicing and, at the same time, discharge all the material on the dam without rehandling. This condition made sluicing comparatively simple. The largest rock carried by the sluices was about 1 ft. square by 3 ft. long, and a depth of water of between 6 and 8 in. was sufficient to transport rocks of this size on a 6% grade. One side of the valley was shale rock in ledge formation with only from 4 to 6 ft. of glacial drift on top, the other side being composed of a heavy deposit of glacial drift and clay with many stones and boulders mixed through it. It was not difficult, therefore, to get nearly any desired proportion of coarse and fine material.

The shale was drilled and shot with a light charge of dynamite so as to shake it up a little without throwing it down. This made sluicing easier and, by so doing, it was quite practicable to carry a 40-ft. vertical face in the sluicing pit. It also hastened the work to shoot the clay. In either material, a very small crack was all that was necessary to give the water a chance to do its work.

All material was conveyed to the dam in wooden sluices, 24 in. wide and 18 in. deep, built on a 6% grade, and placed so that they discharged the material at the extreme outer edges of the embankment on both up-stream and down-stream toes. The very coarse material, composed of large stones and rock, remained where it was discharged from the sluice-box, while the intermediate grades, between very coarse and very fine, were deposited at varying distances from the sluice, depending on the quantity of water used in sluicing and the elevation of the central pool for controlling the deposit of core material. The fine sands produced by the sluicing would build up on a very steep slope at the edge of the pool for a height of 2 or 3 ft., then slough off and slide into the soft core material. This operation was repeated to a greater or less degree during the whole sluicing period. The central pool was usually from 1½ to 2 ft. deep, and at these depths it was large enough, for two-thirds of the height of the dam, to allow quite complete sedimentation so that there was very little loss over the waste weir provided to carry off the sluicing water.

The width of the core at the base was a little less than one-third the total width of the section, decreasing as the height increased. The proportions gave room for a sufficient bulk of coarse material to hold the core in place. It is the writer's opinion that this thickness of the core is greater than was necessary and in the future he would be inclined, under like conditions, to reduce its cross-section materially on the theory that less thickness is ample for water-tightness and that the increased quantity of coarse material is an added security against sliding. Under some conditions and in some locations, depending


About 710′
20′
13
Spillway Level 1620 Ft.—10 922 Millions of Gallons Stored
Stone and Very Coarse Sluiced Material
3 to 1 Slope
4 to 1 Slope
Coarser Sluiced Material
Fines–Sluiced Puddle
93
60′ Cleaned to Rock
Datum Level 1540 Ft. Above Sea
Coffer Dam, Placed Before Sluicing Started
Rock
Concrete Cut-off Wall
57
Puddle Cut-off
a
b
c
d
SLUICING PLANT
1350 H.P. Water Tube Boilers.
Two 14 In. Two-stage Centrifugal Pumps Driven by a 1000 H.P. Steam Turbine, Delivering 11 000 Gal. per Minute Against 250 Ft. Head when Operated in Parallel or 5500 Gal. Against 500 Ft. when Operated in Series.
Surface Condenser in Suction Line to Pumps.
All Seams in Cut-off Trench Grouted to Refusal Under 50 to 70 lb. Pressure
CAMBRIA STEEL CO.
QUEMAHONING RESERVOIR
Steel Pipe
Rock

Concrete Couduit, 30 Ft. Wide 20 Ft. High, for Passage of Stream while Dam is under Construction; Upper End Afterward Closed by Gate-house and Pipe Laid Through it.
13 250 Cu. Yd. Concrete in Conduit.

FIG. 13.

Mr. Curtis. largely on the kind of material, it might be best to steepen the slopes, omitting the 4 to 1, for instance, and using 3 to 1 for the full height of the dam. A steeper slope than the latter does not seem to be desirable.

In support of this statement the writer will mention the Salt Lick Dam, of the Johnstown Water Company, which has 3 to 1 slopes both up stream and down stream. It was built at the same time and in the same manner as the Quemahoning Dam. Frequent pipe tests were made of the core, as the work progressed, in order to obtain some idea of the solidity of the material. As mentioned previously, from 30 to 35 ft. was the average depth reached without driving, and 60 ft. by driving. At corresponding depths the samples were usually the same and about the consistency of stiff putty.

No tests were made for fineness of the core material, but chemical analyses and determinations of specific gravity, the results of which are given in Table 3, were recorded.

TABLE 3.—ANALYSIS OF CORE MATERIAL, QUEMAHONING DAM.

No.	Percentage of loss on ignition.	Percentage of iron peroxide.	Percentage of alumina.	Percentage of silica.	Percentage of calcium carbonate.	Percentage of magnesium carbonate.	Specific gravity.
39	8.06	7.08	17.36	61.62	Lime	3.10	2.45
40	7.70	8.34	13.08	68.52	0.68	1.15	2.63
41	6.99	8.24	14.68	67.56	0.76	1.41	2.55
42	5.16	5.96	10.84	76.52	0.36	0.68	2.44
43	7.14	6.96	13.44	70.52	0.60	0.82	2.59

It should be noted that the specific gravity of the material varies as much as 8 to 9%, and is less than the result given by Mr. Hazen, which was 2.65.

It may be of interest to give the results of the analyses of two samples of sluiced material from the Sánta Amoro Dam, São Paulo, Brazil, sent to the writer by the late James D. Schuyler, M. Am. Soc. C. E., in 1918. The analyses are given in Table 4.

TABLE 4.—ANALYSES OF SLUICED MATERIAL, SÁNTA AMORO DAM.

No.	Percentage of loss on ignition.	Percentage of iron peroxide.	Percentage of alumina.	Percentage of silica.	Percentage of calcium carbonate.	Percentage of magnesium carbonate.	Specific gravity.
21	13.82	10.85	36.08	38.68	None	0.22	3.94
22	14.70	8.68	37.80	37.20	"	0.17	3.90

Mr. Curtis.

There is a marked difference between the analyses of the material in the Quemahoning Dam and that from South America. The extremely high specific gravity of 3.90 and 3.94 is, the writer assumes, rather unusual, although sluiced material from the Nuunau Dam, Honolulu, Hawaii, showed a specific gravity of 3.11. The writer is indebted to Mr. Schuyler also for this sample.

The solidity and compactness of the Quemahoning Dam may be indicated by the amount of settlement after completion. To provide for any possible shrinkage due to settlement, an additional height of 3 ft. was added to the dam. Careful levels were taken frequently along both the up-stream and down-stream edges of the top width and after a period of nine months from the date of completion, the average settlement was not more than 9 in., or less than 1% of the height. From this, it is apparent that although the core material was very soft when placed, it later became a rather compact mass of plastic clay. It is quite probable that the dry material, placed by hand on top, settled into the semi-liquid layer of core material, absorbing the water it contained, so that the whole mass became stable.

The writer has referred at some length to his own experiences with hydraulic-fill dams because the results obtained coincide so well with, and support, the author's statements. The Quemahoning and Salt Lick Dams show that dams of considerable height can be built successfully by the hydraulic-fill method.

The fineness of the core material and the size of particles that should be wasted depend very largely on the character and quantity of coarse material available. It would seem to be desirable to sluice as much of the coarse material as possible and deposit it at the outer edge of the embankment. In this way the medium grades of material are carried toward the central pool by the water, becoming thoroughly compacted in the operation, and making a firm unyielding mass with practically no settlement after completion. Throughout the construction of the Quemahoning Dam, all the area from the outer edge to the central pool was firm enough to walk on without sinking in.

From what the writer has seen elsewhere, and from his own experience, he is satisfied that where conditions warrant its adoption, hydraulic-fill dam construction is practical, economical, and a very satisfactory type of earth dam. It is a type of construction which, although apparently simple, requires close attention to details.

Mr. Chase.

RICHARD D. CHASE,* ASSOC. M. AM. SOC. C. E. (by letter).—After spending about two months at the San Pablo Dam, the writer became so impressed with the small labor and plant cost, that he thoroughly believes the hydraulic-fill method is worthy of being adopted in the East. It is perfectly true that failures of these dams have occurred, but the number of successful ones is sufficient to indicate that success

* New Bedford, Mass.

Mr. Chase. is not due to chance, and Mr. Hazen gives good theoretical and practical reasons for the failure or success of such dams.

When one observes the light soil and the soft, friable, quickly disintegrating rock which are the only available materials at San Pablo and Calaveras, one can realize that the hydraulic-fill method would be most applicable to the heavy glacial drift of the northeastern part of the United States.

With the present unsatisfactory labor market, particularly for heavy manual labor, any method of construction that will decrease the number of men on a piece of work deserves more than usual consideration. It was hard to believe that the half dozen men on the dam and the few on the hillside were putting in 100 000 cu. yd. per month. There was also a comparative absence of expensive plant. It is true the situation was most favorable, with the borrow-pits above and directly adjacent to the ends of the dam—an advantage hardly probable in the East—but even with preliminary handling the advantages are so obvious that serious consideration should be given to the hydraulic-fill method for any earth dams.

The writer believes that this paper will clarify the theory and practice of the hydraulic-fill dam so that it will not be treated as a doubtful experiment, with the chances of failure outweighing its obvious advantages, and that by following the theory laid down one can feel that he is proceeding along sure and definite lines, with a method provided for testing the results obtained so that the chance of failure is eliminated.

Mr. Elliott. G. A. M. ELLIOTT,* M. AM. SOC. C. E. (by letter).—This paper is timely in that it brings to a head and assembles, where it is readily accessible, the results of present experience in hydraulic-fill dam construction. The fundamentals of this type of dam, so clearly set forth by the author, have too often received little consideration. This method of dam construction is attractive on account of its comparative cheapness, and has been adopted for that reason in some instances, when, with the same materials, another construction method would have been more suitable. The hydraulic-fill dam is not to be regarded as a general type, to be built under all conditions of earth-fill, but should be used only when all the facts make it advisable.

It seems to the writer that if the proper relation between width of toes and core is maintained, the grain size of the core material will automatically take care of itself. For instance, if the material used is of a hard rock base, such as a gravel deposit, it will probably not break up under handling sufficiently to give a preponderance of very fine core particles, and naturally a narrow core and comparatively wide toes will result. An example of this is to be found in the Paradise Dam near Chico, Cal. The material in that locality is of volcanic

* San Francisco, Cal.

Mr. Elliott.

origin, and some difficulty was experienced in obtaining a sufficient quantity to insure an impervious center. During construction the drainage through the toes was of such proportions as to be alarming to one not familiar with the hydraulic method. This dam has been completed and in use for three years with excellent results so far as tightness is concerned. In the case of softer materials which, in handling, will break up to such an extent that an excessive quantity of fines is delivered to the "core-pond", the wastage of the surplus puddle necessary to maintain a narrow impervious center will result in automatically discarding very fine particles. A core-pond width of not more than 20% of the total thickness of the dam at all elevations is sufficient for water-tightness, is not too narrow to interfere with ease of construction, and, with proper care in distribution, there is no danger of tongues of coarse material encroaching on the impervious area. To hold to this width in dams built with the comparatively soft materials which must sometimes be used, will necessitate wasting considerable material. This increases the construction cost. It is difficult to persuade engineers of the necessity for wasting material which has cost a great deal of money to break down and transport a considerable distance. Until this fact is fully recognized, failures of hydraulic-fill dams will continue to occur.

It is important not to exceed the predetermined boundaries of the core-pond at all stages of construction. When the fill is carrying an excess of core material there is a tendency to retain all of it, on the theory that even though the clay overlaps the dry toe area, it can be displaced later by the use of other material carrying an excess of rock. In some cases this displacement is attempted by dumping dry fill along the edges of the pond. At best, it is uncertain just what happens under this treatment. It is the writer's opinion, after some experience with the latter method, that although, of course, the dry material does displace a certain quantity of liquid clay, the character of the resulting fill is not as good as the dry banks laid down by sluicing. If clean rock is used, the interstices are filled with liquid clay, and the hydrostatic pressure may be transmitted through the new fill to the original dry banks. The new fill is lubricated, and if the internal pressure is sufficient to move the dry toe, the mass of lubricated rock, instead of adding stability, will move almost as readily as the core. If a mixture of earth and rock is used there may be danger of masses of this material trapping large pockets of clay, which, in effect, provides points of lubrication. Well-drill explorations of an embankment made by combined hydraulic placing and dry dumping will not give complete data, as the operation of boring the well grinds up the fill into small particles. Even if this character of exploration does indicate that the coarse material has apparently displaced the clay, it tells nothing in regard to the fact that the voids in the rocks may be filled with liquid core.

Mr. Elliott. Exploration by borings in the up-stream toe of the Calaveras Dam made during construction indicated a solid fill in locations in which the core had been replaced by dry fill. In the light of later events, the writer is convinced that the mass of coarse fill, placed by wagon in and along the edge of the core-pond, was permeated by the semi-liquid clay core, and that it added little or nothing to the stability of the structure. The action of the down-stream toe, which, although not as thick as the combined up-stream toe (placed by hydraulicking and wagon), was built of dry material dumped from cars on trestles, bears out this conviction. This embankment was built up vertically, that is, the inner face did not encroach on the core-pond to any great extent with increased elevation, and the material was not placed in the core itself for the purpose of forcing it back. Under this method there was no opportunity either for trapping the clay masses or for the voids in the dry bank to be filled with wet core material. In spite of its smaller section, this embankment did not move horizontally. Mr. Hazen's quicksand theory would apply directly to that portion of the dry embankment built by displacing the core by dry dumping.

A semi-hydraulic-fill method has been used with success on the Pacific Coast in cases where the material available consisted of soft rock which would not lend itself to economical spreading and rolling and would break down badly with water transportation. Briefly, the material is transported in cars and dumped on the up-stream and down-stream embankments, leaving a predetermined distance between for the core. This space is filled with water, and a pump mounted on a barge is used to jet down the exposed inner surfaces of the banks. By using this plan the core is under absolute control at all times. The dry banks are loosely placed, and consolidation occurs in time through natural settlement. In the case of small dams which might be built in one season and filled immediately thereafter, there might be some objection on the score that the embankment had not consolidated sufficiently, but on large dams requiring several years to construct the settling action due to rain and time would offset this feature. The down-stream embankment of the Calaveras Dam was built in this manner, and showed no signs of settlement beyond two years after placing. The Haiwee Dam built by William Mulholland, M. Am. Soc. C. E., in Owens Valley, California, was successfully constructed in this way.

Mr. Howells. J. M. Howells,* M. Am. Soc. C. E. (by letter).—Engineers are indebted to the author for an instructive analysis of the fundamental principles involved in hydraulic-fill dam design and construction and as a contribution to the discussion, the writer presents only minor variations of viewpoint from those taken by Mr. Hazen. One of these differences is in the matter of homogeneity of core material, for the

* San Francisco, Cal.

Mr. Howells.

writer has noted that almost invariably toward the finish of a dam, as the top approaches the form of an elevated canal, the construction water flows so rapidly that it carries off core material needed for water-tightness. Also, at this stage of the work, as the hydraulic fill is delivered at both the up-stream and down-stream faces, it has been difficult to prevent the deposit of coarse porous sand "tongues," which often reach entirely across the central pool, thus tending to loss of water-tightness.

The writer wishes to present one phase of Mr. Hazen's paper concerning which confusion on the part of the members of the Society seems to prevail, that is, the simple question as to what constitutes an hydraulic-fill dam.

At a meeting of the San Francisco Section of the Society, a committee was appointed, composed of Mr. Hazen, C. E. Grunsky, M. Am. Soc. C. E., and the writer, as Chairman, to "formulate and present to the Section, the definition of the term 'hydraulic-fill dam' ".

As this type of dam is a Western development, it was considered appropriate that the San Francisco Section should presume to appoint a committee to formulate this definition. The report of that committee follows:

"*Definition of an Hydraulic-Fill Dam.*—The hydraulic-fill dam method of constructing earth dams implies a conveyance of material to its place in the structure by flowing water, but the material may be brought to the structure by mechanical means, providing it is properly washed and segregated while it is in the process of being deposited.

"Any dam constructed throughout by this method is an hydraulic-fill dam.

"The method further implies more or less separation of the material into two distinct classes, each deposited in definite locations within the cross-section of the dam—the coarsest, most gritty, and porous conveyed and deposited by running water and constituting the outer portions of the dam, to secure stability; and the finest and most water-tight material deposited under standing water and constituting the central portion, insuring water-tightness.

"Whenever an appreciable part of a dam is built by the dry-fill method and a part by the hydraulic-fill method, the structure is no longer to be classed as an hydraulic-fill dam, but is a combination structure—a loose rock dam with hydraulic-fill core, a dry-fill earth dam with hydraulic-fill core, etc., as the case may be."

According to this definition, the Calaveras Dam is not an hydraulic-fill dam.

In referring to the mysterious action of the material in the toes of the Calaveras Dam, its lack of stability before the accident, and and its tendency to stabilize after the accident, one might look to a fact not brought out clearly enough by the author, namely, that, as previously stated, the Calaveras Dam was in no sense an hydraulic-fill dam. It was started as such, and about 60 ft. of the base was built in

Mr. Howells.

that manner, and this part was found to be intact after the disaster. It is important to bring out this fact, as many of the technical papers have called this structure an hydraulic-fill dam.

Above the 60-ft. base, the dam was composed of a pumped hydraulic-fill core of slimes, built between toes, like dry railroad dumps, with no washing of the latter to carry away the native slimes which when wet are inherent in their masses. Neither was any apparent effort made to build these dry fills in thin horizontal layers, a practice considered as standard for earth dams other than those built by the hydraulic method. If there were any layers in these toes, they were on a slope of about 1½ to 1, and resulted from the sliding over-dump of the dry material. As this dumping of the toe material progressed, great porosity or looseness of the mass resulted, and the writer has been told on good authority that the puddle-core material often found an outlet by irregular channels through this loose dump to the faces of the structure. Thus, while settlement was in progress, lubricated planes were being arranged to take part later in the ultimate disaster.

That the slip did not take place on the down-stream face which was steeper and theoretically less stable, was probably due to its opportunity for drainage. On the up-stream face, both the reservoired water and concrete facing interfered with such drainage. This, together with the sloping structural planes of deposit (already in movement as they successively settled), must explain to a great degree the point mentioned by the author in his endeavor to account for the low coefficient of friction in otherwise fairly stable material.

Had the Calaveras Dam been a true hydraulic-fill dam, its toes would have been brought up in thin vertical increments from which all slimes would have been washed and stability insured.

The writer agrees with the author on the possibility of too broad a core being left where the true hydraulic-fill section ended, and that this might add to the tendency of the face to slide or to transmit pressure dangerously close to the faces; for in an hydraulic-fill dam of the height proposed, stability could be insured only by great width of base of both toes. This is shown by the accepted formula in which coefficient of friction and weight of mass rather than percentage of voids controls.

In this connection there is grim comfort to the builder of hydraulic-fill dams or any of their variations, namely, that they are never so weak as during construction, for at that time they are virtually two slender dams storing between them a heavier-than-water fluid. Therefore, if they stand under these conditions, they have been tested with a liberal factor of safety and are ready to assume their greater responsibility of storing a great volume of water.

As for the author's suggestion to waste core material finer than a given size: It often happens that this fine material is needed as against the porosity of the river bed itself above the dam, and in cases of dam

Mr. Howells

construction within an area of about 200 000 sq. miles in the Northwestern United States, where porous lava flows cover almost the entire country, this waste would be useful. The Big Meadows Dam is within the area mentioned, and its successful construction and seven years of operation argue for the special adaptability of the hydraulic-fill dam to such foundation conditions.

The author places a great deal of stress on his void theory and, within the limitations which he places around it, there is no doubt but that the percentage of voids in the puddle core can be studied profitably, but, after all, no reliance should be placed on the puddle core for stability. It has another office.

As to the toes of hydraulic-fill dams, voids are desirable rather than otherwise, and should add to rather than detract from the stability of the entire structure.

In the original conception of the hydraulic-fill dam it was hoped, more often than later proved feasible, to secure a gradation of material from coarse at the faces to fine in the center, similar to that described in the paper* by the late James D. Schuyler, M. Am. Soc. C. E. This would insure greater voids toward the faces, better drainage, and less chance for water pressure as the faces of the dam are approached, and any drainage from the puddle core, or perhaps through the dam after construction, loses the possibility of destructive pressure by successively entering more and more porous beds of sand, gravel, and rock. Thus, the pressure diminishes and drainage flow begins so far within the structure as to be inert as against destructive effect. This was noticeable for its absence in the railroad-dump type of toes exemplified in the Calaveras Dam.

As to the use of rock-fill in the toes, this, at times, seems to be excellent practice and became a part of the design of the Big Meadows Dam, in the construction of which the problem of stability and water-tightness presented itself in a peculiar form. That is, it was necessary to connect the central puddle core with a water-tight layer placed over the stream bed for approximately ¼ mile up stream from the dam. By first placing massive basalt rocks for stability into the proposed up-stream toe, on a gritty foundation of native boulders, gravel, and sand, it was possible later to pass fine hydraulic fill through this basalt rock-fill and onward up stream to line the old river bottom and also to secure connection of this fine river lining with the puddle core of the dam.

In conclusion, the writer wishes to state that he agrees generally with the author's final summary.

Mr. Henny.

D. C. Henny,† M. Am. Soc. C. E. (by letter).—The author has given with his usual clearness the reasons which render the use of very fine material in the core of hydraulic-fill dams objectionable. Dams

* *Transactions*, Am. Soc. C. E., Vol. LVIII (1907), p. 196.

† Portland, Ore.

Mr. Henny. with such cores, however, have been built and have not failed, so far as the writer is aware, after construction had been carried to completion.

An interesting and perhaps important question is the condition of the core after several years' use. The author points out that a reduction of water content from 70 to possibly 50% may proceed by vertical settlement with relative promptness. There comes a point, however, when the movement of water through the mass becomes extremely slow, while a condition of horizontal pressure transference and lack of stability still exists. Full stability may be reached after voids have been reduced to about 40 per cent.

It was pointed out that a void reduction from 50 to 40% means that 6 cu. yd. would become compacted to 5 cu. yd. Whether such a core in a dam which periodically or permanently stores water, will ever reach this degree of compactness may well be doubted, as the repeated watering which the core receives, will permanently prevent complete consolidation on the water side, and this retarding effect may be carried far into the body of the core.

It is quite probable, however, that the down-stream side and the upper portions of the core will undergo a drying out process through slow evaporation, and thus it may be expected that the core, especially near the top, may show a condition of supersaturation on the up-stream side below the level of high water, gradually changing in a down-stream and upward direction to one of compactness.

The compacted or dryer portion must have lost volume. The resultant condition in the core of a dam like the Necaxa has never been examined by shafts, so far as the writer knows. Volume reduction may have taken place through vertical settlement, but after the water content has been reduced to some limit like 50%, it is not improbable that further drying will produce vertical cracks, and such cracks in themselves would act as drains and thereby expedite the drying process.

If a dam with an hydraulicked clay core was to store no water for a long series of years, it is conceivable that cracks might reach through the entire core, at least in the upper portion of the dam, a condition which if not immediately dangerous in view of the high power of resistance to erosion which dry compacted clay possesses, is neither desirable nor permanently safe. Such a condition may be regarded as another important objection to the use of clay in cores, at least until actual inspection of clay cores in existing dams proves the foregoing conception to be erroneous.

The writer wishes further to call attention to the effect which the rise of water against a dam with a clay core, either during the construction period or thereafter while the core is still semi-fluid, may have on the possibility of a slide toward the reservoir.

Mr. Henny.

If Fig. 14 represents the cross-section of a completed dam, of the dimensions indicated, with a semi-liquid core, and the weights of material may be assumed to be similar to those found by the author at the Calaveras Dam, as follows:

Weight of core material with about 50% water content	100 lb. per cu. ft.
Weight of material in outer sections.....	120 lb. " " "
Weight of material in outer sections when dry	110 lb. " " "
Weight of material in outer sections submerged	73 lb. " " "

then, with water in the reservoir at or below the base, the weight on the base, *AB*, of the up-stream toe section per linear foot of dam is 400 tons. The horizontal outward pressure of the core is 202 tons and the sliding factor is approximately 0.5.

As the water in the reservoir rises there will be created a horizontal counter pressure tending toward a reduction of the sliding factor. At the same time, however, the weight on *AB* will become reduced through submergence of the material, tending to increase the sliding factor. The combined effect is shown for various heights of reservoir water to the right in Fig. 14.

It will be noted that the danger of sliding increases 30% as the reservoir fills to 40 ft. above the base of the dam, after which it decreases. This indicates the danger which may exist in allowing the reservoir to fill during its construction or even some time after its completion.

In the case considered, with reservoir empty, a sliding factor of 0.50 is calculated in a dam with a water slope of 3 to 1. The author argues that such a sliding factor will be obtained with a water slope of 2 to 1.

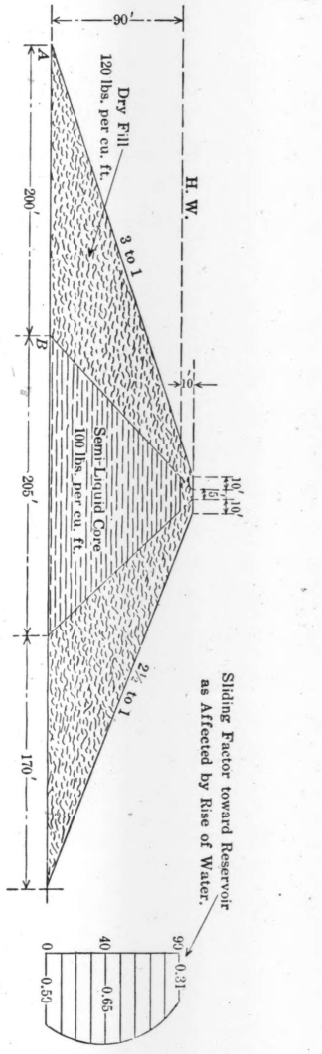


Fig. 14.

Mr. Henny. This is, however, under essentially different conditions. The difference lies partly in that Mr. Hazen bases his statement on equality of weight of toe and core material instead of the weights of 120 and 100 lb. per cu. ft., respectively, herein assumed.

The principal difference, however, lies in his assumption that the weight of the entire triangular mass in the dam up stream from the top contributes toward resistance to sliding. This can be true only if the core material, or horizontal layers of it, do not extend under the slope. Where this does occur, the author mentions for the Calaveras Dam sliding factors of 0.20 and 0.22, which are obviously average factors for the entire toe base and not the much higher factor in the toe material considered by itself, which, as the writer sees it, is controlling.

Reference in this connection has been made in a brief discussion of this problem* in which the writer argues, in connection with the Calaveras Dam, from such information as was available on the ground and as was subsequently furnished by the author, that at the time of the Calaveras slide the sliding factors in the base of the toe material may have been close to 0.7 toward the reservoir and 0.5 away from it. These values although they cannot be claimed to be accurate, due to lack of definite information regarding distribution of materials in the dam, at least explain satisfactorily why sliding took place in an up-stream and not in a down-stream direction, and that so far as information goes, 0.7 may have been approximately the coefficient of friction in the toe material.

The writer's analysis may be in error in so far as he assumes the weight on the toe base to be that of the material vertically above it. This may be reasonably correct for a dry-built dam, but in the case considered may be in error in view of the tendency of the entire outer section to slide toward the reservoir along the up-stream side of the core. To test this point a brief analysis was made of the case illustrated in Fig. 14. Neglecting the weight of the small part of the dam above the high-water line and considering only the part of the dam up stream from the vertical, *CD* (Fig. 15), the following values result:

Weight of toe section	618.3	tons
" " core "	222.8	"
Total weight	841.1	"
Load on core base 4.5×99	445.5	"
Load on toe base	395.6	"

The weight of toe material vertically above the toe base is 400 tons, so that in this case the previous assumption is practically correct, as is also indicated by the loading diagram in Fig. 15.

* *Engineering News-Record*, March 6th, 1919, pp. 487–488.

Mr. Henny.

This close equality, however, may be somewhat accidental. Submitting the case of the partly completed Calaveras Dam, before the slide, on the basis of the section shown in the writer's article previously quoted, to a similar analysis, the figures stand as follows:

Total weight of up-stream section	1 787 tons
Weight on core base	1 304 "
Weight on toe base	483 "
Weight of toe vertically above toe base	548 "

There is considerable difference in this instance. If the latter analysis is the more correct, it would seem that a large part of the weight vertically above the toe base is transferred to the core base; also that the coefficient of friction at the time of sliding may have been in excess of 0.7.

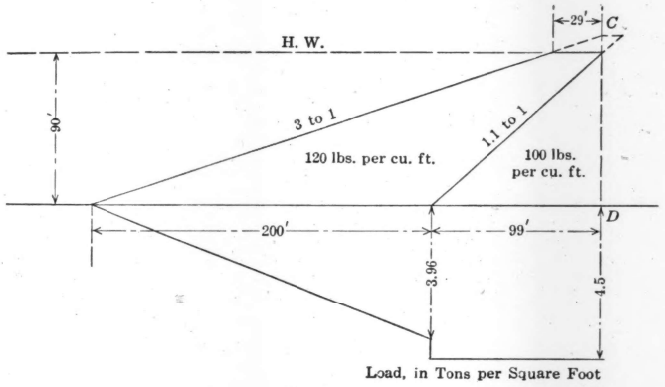


FIG. 15.

The uncertainties unavoidably involved in dealing with a structure of which the interior condition is so little known preclude the derivation of any definite friction coefficient and show the necessity of adopting some factor of safety, as argued by the author.

The author by an interesting experiment at San Pablo Dam has found coefficients of friction of 0.45 to 0.55, and for harder material of 0.70. He found further that values with pressures lower than those due to a depth of fill of 200 ft. were found to be distinctly larger, and he considers the materials at San Pablo and Calaveras as quite similar. Considering that the toe pressures at Calaveras before the slide were due to an average height of fill of less than one-third of 200 ft., there appears to be nothing in the result of the experiment disproving a possible friction coefficient at Calaveras of 0.7.

Mr. Henny.

The limitation as to fineness of material which is to form the core is obviously a measure of safety, and the author's definition of the limit is especially welcome. It is probable also that it can often be applied in construction without thereby rendering the hydraulic process uneconomical. Undoubtedly, cases will arise, however, where with an abundance of clay and intermingled coarse material a shortage of the finer particles approaching the 0.01-mm. size will occur, and that a clay core must be depended on for tightness.

Safety and economy will then combine in suggesting a far thinner core than has been commonly used heretofore. The author mentions testing out a narrow pond at the San Pablo Dam, so narrow indeed that hardly more than a muddy creek remained in the middle of the dam. The writer's experience leads him to think that under such extreme conditions the passing of coarse sand layers or lenses through the entire core is highly probable, due to changes in the material delivered from the pit—an undesirable although not necessarily dangerous result. The paddling of pond material described in connection with some dams has been practiced in the past to destroy such stratification. The writer, however, has found it to be practically impossible to mix pond material in that manner where the sand layers were of any appreciable thickness.

If a clay core must be used and sufficient graded coarse material can be sluiced to hold the core material and form the outer section of the dam, the proper solution may be a core of no greater thickness than can be hydraulicked without danger of sand layers passing through, the remainder of the clay to be wasted. The design of the Calaveras Dam is in accord with past practice, yet there appears to be no reason why a similar dam should not have been entirely tight with a core 100 ft. thick at the base instead of over 400 ft., and it is certain that with such dimensions the slide could not have occurred.

The author's conclusion that with a coefficient of friction of 0.5 in the toe material and with equality of unit weight of toe and liquid core, an outer slope of 2 to 1 will provide a factor of safety of 1, implies, as previously stated, that a liquid core in a solid mass or in horizontal layers does not extend under the slopes. If the core occupies one-third of the mass, a factor of safety of 1 under the same assumption of a coefficient of 0.5 would require a slope of 3 to 1 instead of 2 to 1, a distinction which it may be of great importance to bear in mind in view of the fact that hydraulic-fill dams, in all cases with which the writer is familiar, as well as dams with the narrowest feasible clay core, will have a considerable quantity of core material extending under the slopes.

Referring to the claims that considerations of stability require hydraulic-fill dams to be given flatter slopes than other earth dams, the writer believes that although from the previous arguments it may

be concluded that such claims are valid in the case of cores of large dimensions consisting of very fine material, they lose force where the clay core can be made thin and the material for the outer sections is reasonably coarse, and such claims appear to be entirely invalid where the core is composed of larger particles of about the limiting size of 0.01 mm. suggested by the author, since in that case the core has self-draining properties and becomes promptly as hard and stable as material in a well compacted dry-fill dam. Mr. Henny.

A. T. GOLDBECK,* ASSOC. M. AM. SOC. C. E. (by letter).—In his discussion of the measurement of pressures existing in the cores of hydraulic-fill dams, the author mentions a method now being used by the Miami Conservancy District, with a special type of apparatus developed by the writer with the collaboration of Mr. E. B. Smith and others connected with the laboratories of the United States Bureau of Public Roads. It would seem that the author has overlooked the possibilities of this method for, apparently, he is not fully informed as to the degree of accuracy to be expected from this type of instrument. Mr. Goldbeck.

For the benefit of those who are not familiar with the soil-pressure cell mentioned by Mr. Hazen and used very successfully during the past several years in the U. S. Bureau of Public Roads for measuring the distribution of pressures through granular materials, it may not be amiss to introduce at this time a short description of the apparatus, together with illustrations of its application and tests confirming its accuracy.

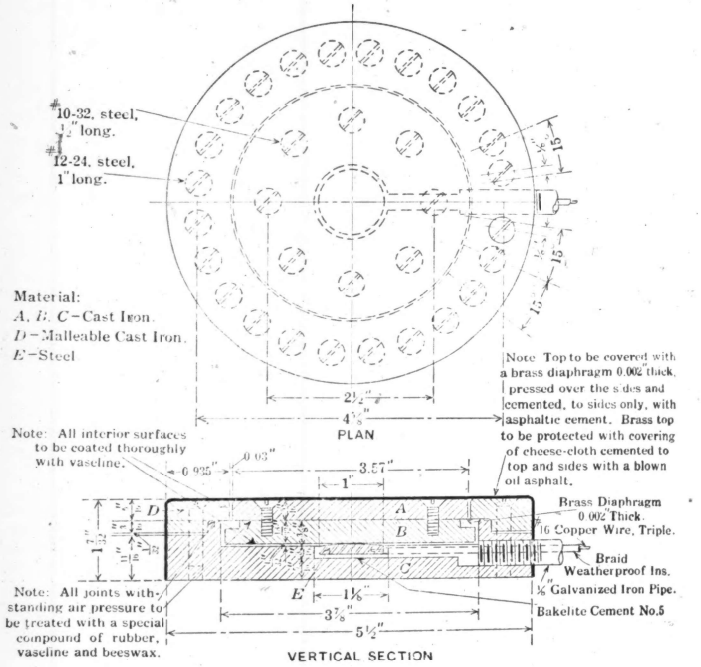
The measurement of pressures transmitted through granular materials has always been difficult, due primarily to the fact that the measuring instrument must be capable of indicating such pressures with only an infinitesimal movement of the instrument itself. The very slightest motion of the weighing face disturbs the soil so that the pressure originally existing may be changed considerably at the instant the measurement is made. Any instrument to be successful for this purpose must be capable of measuring pressures with practically no motion, and the soil-pressure cell mentioned by Mr. Hazen fulfills this requirement.

The principle of its action may best be understood by referring to the cross-section shown in Fig. 16. The underlying idea of this device is to equilibrate the external earth pressure on the weighing face by means of internal air pressure, detecting the instant at which equilibration takes place through the breaking of an electrical circuit within the instrument, and at that instant noting the air pressure required to overbalance the earth pressure.

* Washington, D. C.

Mr. Goldbeck.

Referring to Fig. 16, the soil-pressure cell is seen to consist of a cast-iron base, *C*, 5½ in. in diameter, in the center of which is cemented a steel button, *E*. Bakelite cement is used for this purpose for the double reason that it is an insulator as well as a cement. A thin brass diaphragm, as thin and flexible as a sheet of paper, is stretched across the base, *C*, and is held in place by the malleable cast-iron ring, *D*. This diaphragm is clamped between two cast-iron disks, *A* and *B*, the



DIAPHRAGM CELL FOR DETERMINING SOIL PRESSURE.

FIG. 16.

lower disk, *B*, bearing on the button, *E*. In order to protect the cell from moisture and to prevent the small annular space between Ring *D* and Disk *A* from becoming clogged with soil, the top of the cell is covered with a brass diaphragm like that between Disks *A* and *B*, and the entire instrument is coated with a layer of cheese-cloth cemented with a blown-oil asphalt. Blown-oil is used for this purpose because of its comparative unsusceptibility to temperature change. A galvanized-

Mr. Goldbeck.

iron pipe connects the pressure cell with a small tank of compressed air which is placed in a portable indicating box containing pressure gauges for reading the air pressure within the cell. An insulated electric wire is led through this pipe to the button, *E*, and the pipe itself furnishes the return circuit. A small dry cell and a miniature electric light are the only other electrical equipment required. The indicating box used with this pressure cell is shown in Fig. 17. Proper control valves are also placed in the air line in order to control very delicately the flow of air from the tank into the pressure cell.

Let it be supposed that this instrument is to be used for measuring the pressures back of a retaining wall. The cells are first mounted securely against the wall with their weighing face placed toward the earth fill. It is necessary, of course, to mount the cells before the fill is in place. The pipe connections are led up to an accessible position, and if the cells are to remain in place and the readings continued over a long period, it is desirable to use a double pipe connection, as shown in Fig. 18. Should any water collect in the pipes through condensation or leakage, this fact can be determined very readily merely by allowing the air to flow into one side of the pipe line, the other end being open. If water is present, pressure will be indicated in the air line due to the head of water. The mere fact that water is present, however, does not necessarily spoil the readings of the apparatus, for by the use of a double pipe line, the operator can cause the water to mount into one side of the line while he is taking his readings, which are thus not impaired by its presence.

The operation in taking readings is to open the control valve on the pressure tank, thus allowing the air to flow very slowly into the pressure cell. As long as the cast-iron disk, *B*, in Fig. 16 is in contact with the steel button, *E*, this fact will be indicated by the burning of a small electric light in the indicating box. At the instant, however, that the air pressure within the cell slightly exceeds the soil pressure acting on the outside of the cell, an exceedingly small movement of the plate, *B*, takes place, and it is raised out of contact with the steel button, *E*, the electric light is extinguished, and the operator immediately reads the pressure on one of the test gauges in the indicating box. This records the air pressure within the cell, which necessarily equals the earth pressure acting on the cell. At the same instant the operator releases the air pressure within the cell, for at no time should he allow the air pressure to exceed the soil pressure by more than 0.1 lb. per sq. in., since by so doing the accuracy of future readings would be impaired.

The question of the accuracy of this instrument has been raised by Mr. Hazen, and it may be stated that it was several years before means could be devised for proving the accuracy of the results obtained where measurements of pressure of granular materials were made,

MOUNTING OF SOIL PRESSURE INDICATING APPARATUS.
FOR USE WITH DIAPHRAGM CELLS.

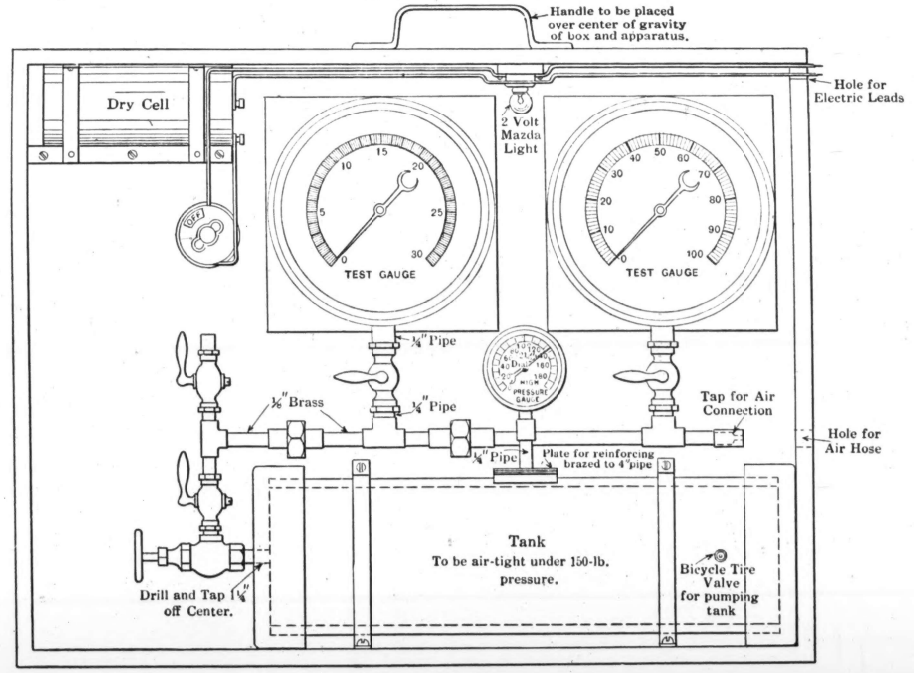
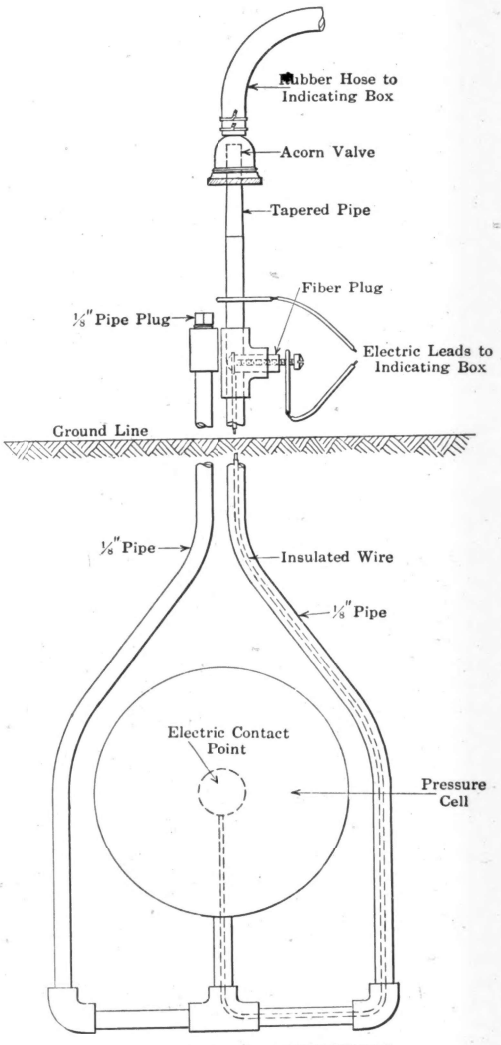

Handle to be placed over center of gravity of box and apparatus.
Hole for Electric Leads
Dry Cell
2 Volt Mazda Light
TEST GAUGE
TEST GAUGE
¼" Pipe
⅛" Brass
¼" Pipe
¼" Pipe
Plate for reinforcing brazed to 4" pipe
Tap for Air Connection
Hole for Air Hose
Tank
To be air-tight under 150-lb. pressure.
Bicycle Tire Valve for pumping tank
Drill and Tap ¼" off Center.

FIG. 17.

Mr. Goldbeck.

Mr. Goldbeck.



PIPE AND ELECTRICAL CONNECTIONS OF SOIL PRESSURE INDICATING CELL.

FIG. 18.

Mr. Goldbeck.

although it was a comparatively easy matter to prove the accuracy of the instrument when used in liquids or semi-liquids.

The method pursued in checking these results for granular materials consisted in measuring the movement of the disk, *B*, during the

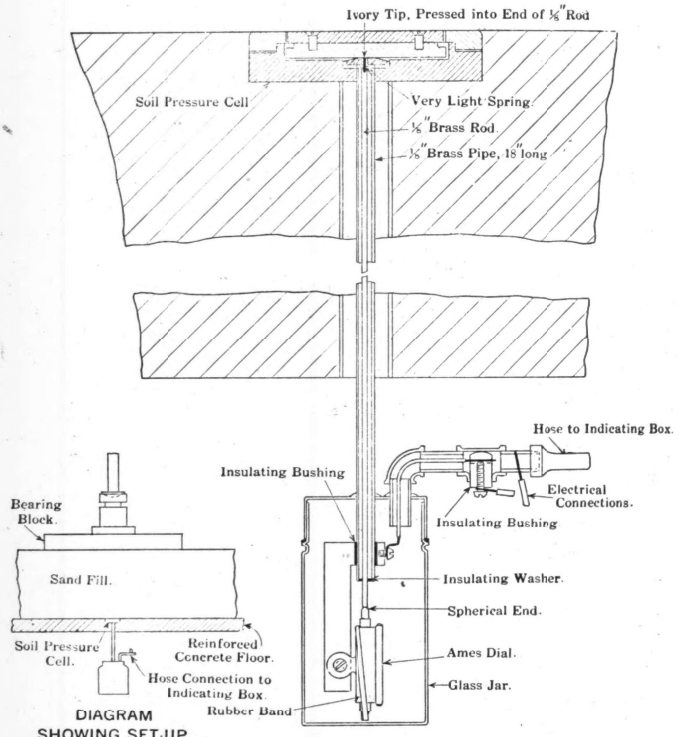


DIAGRAM SHOWING SET-UP OF APPARATUS.

APPARATUS FOR MEASURING ERROR IN SOIL PRESSURE CELLS DUE TO SMALL MOVEMENT REQUIRED TO BREAK ELECTRICAL CONTACT

FIG. 19.

breaking of the electrical contact. For this purpose a special set-up of the instrument was used, as shown by Fig. 19. The cell was placed flush with the surface of a reinforced concrete floor and 3 ft. of sand was placed over the cell. In place of the wire ordinarily used in the pipe line leading to the cell, a brass rod having an ivory tip bearing

Mr. Goldbeck.

CURVES SHOWING MOVEMENT OF UPPER DISK OF SOIL PRESSURE CELL AFTER BREAKING ELECTRICAL CONTACT—30 IN. MOIST SAND FILL.

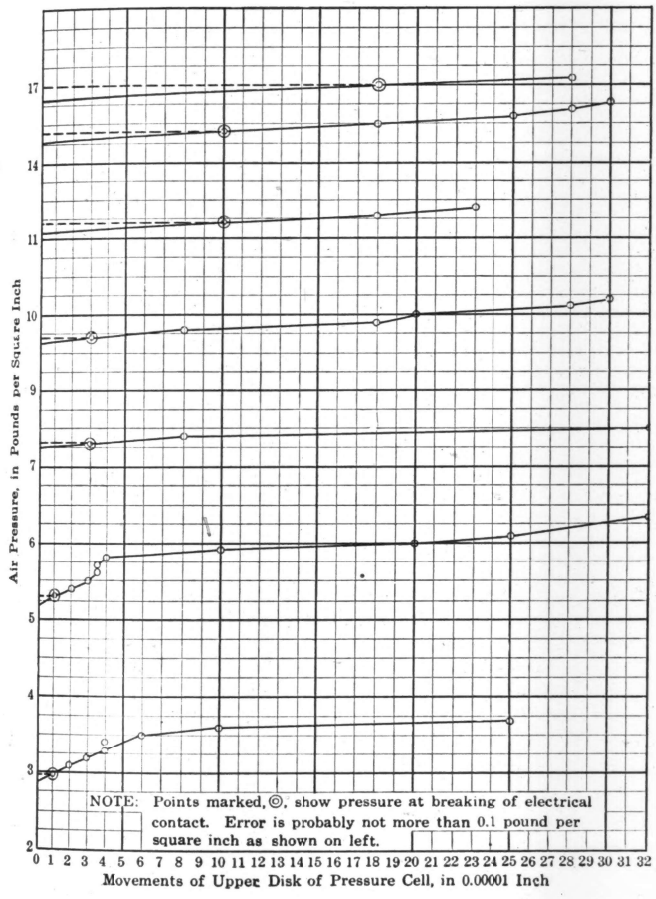

Air Pressure, in Pounds per Square Inch
2
3
4
5
6
7
9
10
11
14
17
NOTE: Points marked, ⊙, show pressure at breaking of electrical contact. Error is probably not more than 0.1 pound per square inch as shown on left.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32
Movements of Upper Disk of Pressure Cell, in 0.00001 Inch

FIG. 20.

Mr. Goldbeck.

on the movable disk of the cell was used. The other end of this rod bore on the plunger of an Ames dial reading movements of 0.0001 in. Concentrated loads were placed on a bearing block resting on the sand fill over the cell and thus definite unit pressures were transmitted to the cell. Air was introduced and simultaneous readings of the air pressure in the cell and movements of the movable diaphragm were recorded at the breaking of the electrical contact, and also after contact was broken.

The curves on Fig. 20 show these simultaneous readings. The double circle shows how much movement took place in the instrument up to the point of breaking the electrical contact, and the circles to the right of the double circle indicate the movements after the breaking of such contact. Before the electrical contact is broken the soil pressure is counterbalanced by the internal air pressure plus whatever pressure exists on the steel button, *E*. After contact is broken within the cell, the air pressure alone counterbalances the soil pressure and, therefore, it is known definitely how much increase in internal pressure is required to produce definite movements of the movable disk.

It will be noted that the movement required to break the electrical contact for external pressures up to about 15 lb. per sq. in. is less than 0.0001 in. and, for low pressures, only 0.00001 in. It will also be noted from the curves that, in general, an increase in pressure of less than 0.1 lb. per sq. in. is required in order to move the disk against the earth pressure 0.0001 in., or less. The small increase in pressure necessary to produce the exceedingly small movement of the diaphragm required to break the electrical contact is the error of the instrument, and this is not more than 0.1 lb. per sq. in., since, in general, a movement of less than 0.0001 in. is required to break the electrical contact. By virtue of the exceedingly small movement required to break the contact, the pressures in granular materials may be accurately determined with this type of instrument.

Numerous applications of this device have been made for measuring pressures transmitted through earth. Thus, the distribution of concentrated loads through sand fills has been determined.* In addition, pressures have been measured back of several retaining walls. At the suggestion of Charles H. Paul, M. Am. Soc. C. E., Assistant Chief Engineer of the Miami Conservancy District, the Bureau of Public Roads conducted an extensive series of tests in an hydraulic fill, in co-operation with the Miami Conservancy District, the test being made in a stand-pipe 41 ft. in height and 30 in. in diameter.† Typical curves resulting from this test correspond very well with those obtained by the Miami Conservancy District in pressure measurements

* "The Distribution of Pressures Through Earth Fills", *Proceedings*, Am. Soc. for Testing Materials, Vol. XVII (1917).

† "Tests to Determine Pressures Due to Hydraulic Fills", *Engineering News-Record*, April 18th, 1918, p. 758.

Mr. Goldbeck.

made with the same apparatus in hydraulic-fill dams.* The Bureau of Public Roads has also made measurements of pressures of concrete against forms with this instrument,† and has utilized it for making measurements of pressure transmitted to the sub-grades of roads.‡

As a result of these tests it was demonstrated that pressure cells of this character may be used successfully for measuring pressures under a variety of circumstances, even in hydraulic fills, for periods up to two years and perhaps longer. The hydraulic-fill pressure measurements demonstrated that material such as is used in the core of an hydraulic-fill dam acts as a perfect fluid during the early stages of settlement, but that it loses its fluid qualities as consolidation proceeds.

As a result of experience with this instrument, the writer believes it to give thoroughly reliable results, not only in liquids and semi-liquids, but even in granular materials such as densely compacted sand. It has also been demonstrated that the instrument is sufficiently rugged to withstand field conditions, and it might very readily be applied in obtaining useful data on many types of construction.

Mr. Dillman.

GEORGE L. DILLMAN,§ M. AM. SOC. C. E. (by letter).—This paper is a distinct addition to the literature on earth dams. It is also timely.

Before entering upon the discussion, the writer would call Mr. Hazen's attention to one misstatement of fact. Messrs. Schuyler and Howells were not, as he states, pioneers in hydraulic-dam construction. The Chabot or San Leandro Dam was built before they were ever heard of, was largely hydraulicked, was for years the highest earth dam in the world, was never a failure or a partial failure, and suffered no accident during construction.

The analyses of earth dams and slides are identical. Slides have occurred during the construction of many earth dams, due to badly drained core material. The writer refers to the Pilarcitos, Necaxa, and Calaveras Dams. Drainage will avoid such accidents, just as it will cure any slide. The hydraulic-fill type makes a dam of each toe, the pressure coming from the core. When the pressure is too great, failure occurs, not failure of the dam as a whole, but failure of a portion of the toe.

The film of water behind any dam, behind any toe, or behind any slide, has only to be thick enough to be continuous in order to transmit the full pressure of its head under, into, and against the resistant

* "Study of Pressures in Hydraulic Dam Cores", *Engineering News-Record*, December 25th, 1919.

† *Proceedings*, Am. Concrete Inst., 1920, or *Engineering News-Record*.

‡ *Public Roads*, April, 1919.

§ San Francisco, Cal.

Mr. Dillman.

material. It acts like myriads of jack-screws to displace material, and it is often effective, as is proven by the movement of the material.

Drainage may be had in various ways. Mr. Hazen's plan for discarding the fine material is one of them; the coarse material drains itself. Building slowly is another, and building the cores narrow is still another. Building the toes to stand the pressure is effective, but is wasteful of material. The writer would suggest strings of tiles. They are cheaper and fully as effective as any other method; the expense of segregation and discard of fine material suggested by Mr. Hazen is avoided and, on the other hand, excess of material in the toes.

The object in an hydraulic-fill dam (or slide) is to break up the continuity of the film of water, thus killing the head and cause of the trouble. Tile drains are exceedingly cheap, and, properly placed, they will not only stop such accidents, but increase the factor of safety in the completed structure by making the "water-tight surface" or "most nearly water-tight surface", farther up stream than it would be without them.

Safety in a dam is not always reached by tightness. Making it tight in the wrong place is often more dangerous than leaks. In 1916, the writer planned, on a pervious foundation, with pervious material, a dam designed to leak. It does leak, some 160 sec-ft., with 50 ft. of water in the reservoir. There was some active objection, first, because it leaked and, therefore, was dangerous, and afterward, that it was too tight and depleted the ground-water below; both these objections came from the same source, which only goes to show that critics, like the poor, are always with us.

If it is ever desired to decrease the seepage, the writer will advise hydraulicking some fine material over the face of the dam. The dam referred to is the Mackay Dam, on Big Lost River, in Idaho. It is an interesting structure, built strictly in accord with the "great hydraulic principle" announced some 17 or 18 years ago, and some day the writer plans to write a description of it.

He can see no objection to fine material in a dam, if it is placed where it belongs; in fact, he rather favors it.

Mr. Muckleston.

H. B. Muckleston,* M. Am. Soc. C. E. (by letter).—This interesting and timely paper calls attention to certain properties of the colloidal clays which appear to be responsible for the observed phenomena. Sands (which term includes all rock powder, no matter how fine) will settle in water in time, and form a stable sediment. The particles of clay in a colloidal condition or state may remain in suspension for an extraordinary time, measured by years in some cases, and form when they do finally settle, not a stable sediment, but rather

* Lethbridge, Alberta, Canada.

a viscous fluid or jelly which retains water to the extent of 60 to 70% or more of the whole, and will not consolidate further no matter how much time is allowed. Extreme pressure, with the addition of some heat, may convert this mass into shale or slate by expelling the water, but neither of these agencies can be active in the case of hydraulic fills, and, consequently, it is doubtful whether such fills ever do become even reasonably dry. Mr. Muckleston.

The writer has met with this phenomenon on two occasions. On one such occasion a shallow lake formed the most convenient supply of water for camp use. The lake was only about 1 ft. deep, with a blue clay bottom, and the water looked much like good gray paint. Filtering was tried, but without success as no water got through the filter. Sedimentation gave no better results, as the only effect of a week's trial was a thin ring film round the top of the barrel, due to evaporation. A 1-pint sample stood in a glass bottle for more than a year and seemed to be just as turbid as when freshly poured in, although the contents at about 20% of the height of the bottle appeared to be a darker color than the remainder. The water was then allowed to evaporate and left a deposit, apparently about 10% of the original volume, of a light gray powder, much resembling talcum powder, in everything except color, and having no gritty feeling between the teeth.

This clay is the result of hydraulic selection. The lake never has more than about a foot of water and, usually, in four years out of six, none at all except in early spring. It has no outlet. In consequence, the finest particles of soil from the surrounding country have been washed down, and as the water, when there is any, is continually stirred up by wind, the finest particles have been gradually brought to the surface with the colloidal particles on top.

The other occurrence was an attempt to back-fill a retaining wall by hydraulic means. The material available was a sandy clay soil containing small pea-size pebbles, but there must have been an undue proportion of the same kind of clay, because the fill never did consolidate and more than a year afterward was removed by boring holes in the face of the wall and allowing it to flow out. The holes were 1 in. in diameter and the wall 30 in. thick, yet the material poured out much like thick molasses.

This colloidal clay may be thrown down by some of the common mineral salts and in this condition looks like boiled starch with or without some color, and when the water is expelled resembles glue in appearance more than the ordinary conception of clay. It has an extraordinary affinity for water, which it absorbs with great energy, rapidly swelling to its former starchy consistency. If given a less quantity, it is much like pitch—tenacious, sticky, and as hard to handle.

Mr. Muckleston.

The alkalies, such as carbonates of sodium and potassium, act in the reverse way to the acids or neutral salts, resulting in a hard impervious substance which is the base of the soils generally known as hardpan or the like.

Mr. Dunham.

H. F. DUNHAM,* M. AM. SOC. C. E. (by letter).—If it is true that "all earth dams leak", some water must have found its way from the construction pool through the toe on the down-stream side of the Calaveras Dam. No movement or bulging of the earth occurred there to show instability. The factor against such movement has been calculated. Still, it would be of interest to learn what the leakage there amounted to, how general it was, and the elevation at which it occurred or was most noticeable. The two sides were quite similar except that the down-stream side had the more abrupt slope, as the description indicates, but in one respect there was a marked difference. On the down-stream side or toe all leakage was free to escape. On the up-stream toe there was, first, a concrete facing and then the water in the reservoir to hold back any leakage and insure more complete saturation of the material filled in by dump-car or by hydraulic methods.

A cross-section of the dam as it was at the date of the failure showing the height of the concrete facing, the water level in the reservoir, and the known-to-be lowest elevation of very soft or plastic material in the core would be of great interest. Of almost equal value would be a knowledge of the rate of construction of the concrete facing and of the rate of increase in depth of water in the reservoir. Together with these items, the known facts about the quantity or extent of the leakage through the down-stream toe should be recorded, as well as the known leakage on the up-stream side. Without all the data, it would seem to be impossible to make reliable calculations as to stability or devise any factor of safety against sliding movements. So far as the paper shows, the items mentioned were not considered together in their relation to the failure.

As a related feature: If it was noticed at any time during the work that the shore lines of the construction pool were irregular and that the bays extended toward the toes on either side, the elevation of such bays and the distance from the center line of the dam to the shore line should be of interest now, especially if it appeared that the outward bulging on the up-stream slope occurred opposite a bay-head where much fine material had been deposited.

The sliding factor or coefficient of friction may have been assumed for movement along a horizontal plane, but if the up-stream toe had become saturated to the level of the water in the reservoir, while the core and other material above were saturated to an unknown depth,

* New York City.

Mr. Dunham.

there would be reason to think that the sliding surface was inclined from the horizontal or toward the base of the outlet tower. How, then, could the real sliding factor be obtained unless that angle of inclination could first be determined? The water in the reservoir would tend to lessen the weight of some of the material in the toe depending, of course, on the extent of the saturation.

Recalling the favorable difference in slope of the two retaining halves or toes of the dam and their great similarity otherwise, there would be some reason to infer that anxiety to complete the concrete facing and to store water led to the Calaveras disaster. If so, there may be a valuable object lesson in the occurrence, in addition to the lessons derived from calculated factors.

Some surprise was caused by the firmness of the core material after the failure. Energy to the extent of many million-foot tons had been expended in that vicinity and in a very short interval of time. A part of that energy may have caused displacement or removal of moisture from the core material. If a large mass of wet concrete falls a considerable distance and stops abruptly, a portion of it will be found to be quite hard and to hold comparatively little water. Had the entire clay core been prepared in some convenient niche in the mountain, several hundred feet above the dam site, and then shot down in 3 or 4 min., the water-tightness would no doubt have been satisfactory. The writer claims no originality for this suggestion. It would be only a feeble imitation of Nature's way in formative processes involving motion and pressure.

Mr. Burr.

WILLIAM H. BURR,* M. AM. SOC. C. E. (by letter).—The hydraulic method for building earth dams has experienced the usual vicissitudes of a comparatively new procedure. The author of this interesting paper has indicated that the method of building an earth dam by hydraulicking the material into place is scarcely more than from 20 to 25 years old. He further observes that the applications of the method have been marked by failures of magnitude, which are due to defective plans of procedure, and not the least prominent among such failures is that of the original Calaveras Dam itself. His paper presents the physical conditions prior to the failure of the dam, which constitute essentially a demonstration of the reasons for the failure.

In order that the fine clayey material of the core deposited by sedimentation may constitute a water-tight mass of material, it must, clearly, be divided into minute particles or grains, and apparently it has been assumed in the past that such material cannot be divided too minutely. At first sight this seems to be plausible enough, but the physical analyses of the core material at Calaveras have shown what might be expected, that is, that the finely divided material with

* New York City.

Mr. Burr. an effective size of 0.001 to 0.003 mm., in which the voids may be from 35 to 40% of the total dry volume, has a capillarity of such intensity and affords such great resistance to the flow of water through it, that it is practically impossible to consolidate it by drainage within a reasonable period of time, if at all. It is Mr. Hazen's opinion that no material of an effective size of less than about 0.01 mm. should be used, that being about the smallest grain that will permit suitable drainage, and he is undoubtedly correct in taking that position.

His conclusions are based on extended investigations of the physical character of the material found at Calaveras after the failure, including not only the voids of the core material but the quantity of water held in them, which had not drained out apparently to any material degree, even after lying in the partly completed core volume for more than a year. The experience gained through the failure of this dam and the demonstrative character of the investigations made prove conclusively what has happened in prior and similar failures. The excessively fine material, separated by the hydraulic process and carried into the pond between the two toes of coarse material, refused to give up its water and consolidate. There is little or no reason to suppose that it would drain within a reasonable period of time, even though the supersaturation is as much as 20 per cent. These observations are probably true, irrespective of the character of the core material, provided the effective sizes of the minutely divided particles or grains are no larger than those found at Calaveras. The intense capillarity existing under such circumstances produces a semi-liquid, the intensity of pressure of which may be measured by the head or distance below the free surface.

The possible results attending such a condition of more or less liquidity are generally recognized, but there are some consequences which apparently have not been fully appreciated, even in the Calaveras case. This supersaturated, more or less liquid, mud is practically the same in kind as the vast mud deposits—some subaqueous and others not—which are found in many places. Piles driven into such material are said to carry their loads by "mud flotation". That material never consolidates, nor is there reason to believe confidently that finely divided material in similar condition will drain or consolidate anywhere, even under conditions like those found in an hydraulic-fill dam.

In the Calaveras Dam, therefore, and in other similar hydraulic-fill dams, the core was at least a semi-liquid mass exerting essentially liquid pressure against the coarse material on either side of it, and this observation is not adversely affected by the fact that the demarcation between the coarse and the fine material is not well defined. The author shows all this in a clear and conclusive manner. The large

horizontal forces thus caused to act against the coarse material of the two toes, however, is not all of the influence of this semi-liquid mass in producing displacement and failure of the two toes. The pool was much wider at the bottom than higher up toward the top of the dam. This produced an overhang of coarse consolidated material which in the Calaveras Dam was considerable. The result was a vertical upward pressure against this overhang, varying with the depth below the surface of the pool. This pressure was great enough to be of material effect in aiding the original failure. As it would tend to relieve the pressure on the bottom of the coarse material near the toe, it should be taken into account in considering the coefficient of friction on that bottom. Mr. Burr.

Again, the larger part of the coarse material of the up-stream face was submerged by the water in the reservoir when the failure occurred. All materials so submerged would of course be diminished in apparent weight by the quantity of water displaced. Inasmuch as all the material of the up-stream toe varies in size from gravel to fragments of rock of considerable size, the water would percolate freely through a large amount of the mass, thus relieving the load on the base of the latter. This, also, should be taken into account in the endeavor to find a coefficient of friction, although the manner in which the mass of coarse and fine granular material is pushed along in such violent displacement, induces a condition quite different from that which exists when a heavy continuous body is, in the strict sense of the word, pushed over a continuous surface in the manner of a true slide. In fact, the motion of such a heterogeneous mass of granular material (including even rock fragments) will partake at some points and, perhaps, at a considerable number of points, at least partly, of the nature of confused rolling and sliding, the resistance to which would be much less than that offered by a more or less rigid mass sliding on an irregular rough surface.

Furthermore, the upward pressure of the semi-liquid against the overhang of the coarse material would tend to produce, and, to some extent, undoubtedly would produce, horizontal or inclined openings in that material closely adjacent to the pool and near its bottom, into which some fine material would find its way and act as a lubricant. Whether any quicksand condition would intervene before the motion of the coarse material begins may well be doubted. The rather indefinite material called quicksand does not take on its "quick" character until the water producing supersaturation takes on motion. It scarcely seems to be necessary to imagine that any such condition acted to aid failure prior to the motion of the coarse material of the toes. The lateral and vertical pressure of the supersaturated core material against the up-stream toe, together with the buoyant relief given to

Mr. Burr.

the volume of the coarse material of the up-stream toe due to submergence, is sufficient to account for all that took place.

A review of the literature of this class of engineering work does not disclose a thorough and systematic examination and control of the core material which has hitherto been placed in hydraulic-fill dams. Various rather crude tests of the consistency or degree of consolidation of this material have been made from time to time during the progress of the work, which are well enough as far as they go, but they have fallen seriously short of the necessities of the case by their failure to give quantitative results. No hydraulic-fill dam can be considered as being safely put in place without careful investigations as to the size and supersaturation of the core material at practically all points and at all stages of the work. It is imperatively essential that the character and deposition of that material be completely controlled. A semi-liquid mud will resist a degree of penetration of a heavy object which may be and usually is highly deceptive as a test of consolidation, as pile-driving in such materials constantly demonstrates.

The procedures at Calaveras and San Pablo, after the failure of the former, as set forth by the author constitute probably the first systematic controlling treatment of core material of this kind. The Calaveras failure, therefore, like many other failures, has proven to be of great value to this important field of earth dam construction. It has shown the necessity of determining and limiting the fineness of core material, the amount of its voids, and degree of supersaturation, to the end that its physical characteristics and the process of consolidation may be completely controlled at all stages of the work.

It appears to the writer that with work performed under effective control, it would be desirable to design hydraulic-fill dams with much less volume of core material than has been used heretofore, that is, with a much less width of core. At any rate, material of excessive fineness, as Mr. Hazen indicates, may be profitably wasted.

Mr. O'Shaughnessy,

M. M. O'SHAUGHNESSY,* M. AM. SOC. C. E. (by letter).—This paper presents an interesting résumé of the hydraulic method of constructing dams. The author has added little that is new on this subject beyond that already furnished by J. M. Howells, M. Am. Soc. C. E., and the late James D. Schuyler, M. Am. Soc. C. E., who made successful applications of this process.

The character of the adjacent materials to be incorporated into the structure dominates the type of dam to be constructed in any particular locality. In Southern California, where Mr. Howells has made successful applications of the hydraulic-fill process, the washed detritus is of a gravelly nature which separates mechanically on deposition in

* San Francisco, Cal.

Mr. O'Shaughnessy.

the dam in the manner prescribed by the engineers, the coarser materials going to the extreme up-stream and down-stream portions and the finer materials toward the core. The porosity of the coarse materials permits drainage of the mass, which leaves the structure fairly safe if work is not prosecuted too rapidly.

An intimate knowledge of the geological and mineral characteristics of the neighboring locality, obtained by study and exploration, is a necessary qualification of engineering supervision, in order to build a hydraulic-fill dam which will be safe.

The terrain of Calaveras Valley, where the Calaveras Dam was built, is of a decidedly clayey nature, as indicated by the material deposited in the completed portion of the dam; and the trouble with that structure resulted from the excessive quantity of clay and the fact that the materials deposited by the hydraulic process, on both the up-stream and down-stream slopes, constituted an admirable core material for an earth dam, if the necessary gravel was added, and this very quality prevented the proper drainage of the structure.

When this work was started in 1913, there was no material in the vicinity of Calaveras Valley suitable for hydraulic-fill dam construction. The gravels and sands along the river-bed carried a large content of clay, which was deposited in the foundation portions of the dam, and as the down-stream and up-stream toes are always kept higher than the central pool, they constituted barriers to prevent the drainage of the pool. The original plans for the dam called for a porous material in both the up-stream and down-stream slopes, and it is very clear, therefore, that the material distribution called for by the plans was not followed in the construction of the dam, due to the excess of clayey contents encountered.

No difficulties of this kind have been experienced in the valleys of Southern California, San Fernando, and other places, where the material washed off the mountains is a detritus from crystalline rocks and includes a very small percentage of clay.

The dominant claim of advocates of this type of dam construction is the cheapness of the hydraulic-fill process, but the writer is advised by G. A. M. Elliott, M. Am. Soc. C. E., that, in the case of the Calaveras Dam, this result was not in accordance with predictions. The initial estimate was about 20 cents per cu. yd., while the actual cost was more than 45 cents, and this was in a time of cheap labor. On taking into account the cost of the pipe and the wear on it, the cost of the pumps and their depreciation, and other expenses, it seems to the writer that the steam shovel and good railway equipment would have been much more economical.

It is a wise precaution to put a rock fill both on the up-stream and down-stream toes of all earthen dams. This construction was used by

Mr. O'Shaughnessy.

the writer about 10 years ago on the Throttle Dam, about 70 ft. high, 18 miles southeast of Raton, N. Mex. This dam had a conventional clay puddle core, 20 ft. thick at the bottom and 8 ft. thick at the top, which was built simultaneously with the rolled earth layers about 1 ft. thick, and well watered, on each side of the core. All stone found in the excavation was placed in the down-stream portion of the dam. The puddle core reached a cut-off wall of concrete, carried from bed-rock, at a depth of about 20 ft. into the structure. The down-stream portion of this dam was always kept higher than the up-stream portion, and notwithstanding the fact that floods as great as 8 000 sec-ft. were anticipated and some damage was expected during the construction (of which the owners were advised), this whole work was completed in a satisfactory manner, at a relatively moderate price, and is absolutely tight and safe, with two large spillways cut into the solid rock at each end of the dam.

On behalf of the City and County of San Francisco, which has now under consideration the purchase of the property, the writer has modified the plans prepared by Mr. Hazen, for the completion of the Calaveras Dam, lowering its height to a crest elevation of 775 ft., with the spillway level at an elevation of 753 ft. He has also provided for a rock toe on the up-stream side, and for bringing the construction up in layers, with heavy rolling and watering, in the conventional manner of constructing earthen dams.

The writer disapproved the suggestion of a dam of the Calaveras hydraulic-fill type at the site of the Alpine Dam in Marin County, California, and its adoption by the Directors of the Marin Municipal Water District would inevitably have resulted in a disaster similar to that which took place at Calaveras. A sane, gravity, arched, concrete dam on excellent bed-rock has now been completed at this site and provision made for raising it at a future date, at one-half the ultimate cost of a hydraulic-fill dam.

The true principles controlling dam design and construction have been clearly elucidated by George L. Dillman, M. Am. Soc. C. E., in his discussion* of the Morena Rock Fill Dam. If those principles are adhered to, either for a masonry, earth, or rock fill dam, there will be no danger of any disaster taking place on the completion of the work.

Mr. Morgan.

ARTHUR E. MORGAN,† M. AM. SOC. C. E. (by letter).—So much has yet to be learned about the principles and facts of hydraulic-fill dam construction that every item of added information is pertinent and valuable. Even expressions of opinion with which one cannot agree are of value when they suggest new points of view or methods of inquiry.

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 52.

† Dayton, Ohio.

Mr. Morgan.

Mr. Hazen's opening statement is: "The hydraulic method of dam construction has great advantages, but too many dams built by it have failed." The searches of the engineers of the Miami Conservancy District have developed only two cases of failure of hydraulic-fill dams, namely, the Necaxa Dam and the Calaveras Dam. In case of other failures of so-called hydraulic-fill dams, it was found that this method of construction was not used. In some cases, efforts have been made to construct embankments by sluicing in materials without any attention to their segregation in the dam, and without the proper use of a central pool.

In the case of the Necaxa Dam, conversations with persons employed there have led the writer to the opinion that the cause of this failure was not so difficult to determine as has been indicated by some of the written reports that have appeared, but rather was due to the failure to observe some of the elementary principles of safe construction. Among other conditions which have been described to the writer as contributing to the failure was that of the up-stream toe. He has been informed that an accumulation of muck and unstable material excavated from the site of the dam was allowed to remain within the cross-section on the up-stream side, and that it was at the location of this pile of débris and waste material that the failure occurred. If this information is correct, it would leave the Calaveras Dam as the single instance of the failure of a hydraulic-fill dam built under careful supervision. However, one failure is one too many.

Mr. Hazen's wide experience with sand filters has drawn his attention pointedly to drainage as one of the methods of consolidation of the core of the hydraulic-fill dam. The writer believes, however, that he is mistaken in considering drainage to be the only, or even the most important, factor in consolidation. About fifteen years ago, in the process of reclaiming wet lands, the writer had occasion to drain thirty or forty shallow lakes in Minnesota. These lakes were located in various parts of the State and under widely different soil conditions. Observation there indicated that the fineness of the material which, according to the author, determines the rapidity of drainage and consolidation, did not appear to be the most prominent factor.

In the western part of Minnesota the glacial drift forming the lake beds is composed of powdered limestone and shale in the form of a very fine blue or yellow clay. In some cases, these clay deposits had become so thoroughly compacted under water that a few weeks after the surface water had been drained away, a road roller could be driven over the surface. In the central part of the State some of the lake beds consisted of a mixture of sand, clay, and organic matter. Here, it was frequently possible to push a wooden pole 25 or 30 ft. into the soft mud in the lake bottoms. In the eastern part of the State, the writer has encountered deep deposits of powdered mica of the glacial period,

Mr. Morgan.

which were so unstable as to flow like water. These materials had remained in place through many centuries, and the powdered limestone deposited as erosion from the surrounding hills became almost as firm as hardpan. Between this and other material which retained almost the consistency of a fluid, nearly every degree of consolidation was encountered. The writer is of the impression that the physical and chemical composition of core material and the shape and specific gravity of the particles have much to do with its rate and degree of consolidation.

The consolidation of core material takes place to some extent by the drainage laterally and downward of the enclosed water, as the author suggests, but to a much greater degree by displacement of the water upward, due to vertical pressures and to the settling of the fine material into the voids. The author mentions this upward movement of the water, but his entire argument thereafter seems to repudiate or ignore it.

In the construction of the dams of the Miami Conservancy District, consolidation of the core material actually is taking place at a fairly rapid rate. It is doubtful whether more than a very small part of this consolidation results from drainage, as defined by Mr. Hazen. A clear distinction should be made between the compacting of material by drainage and that by consolidation through settling under water. In the construction of hydraulic-fill dams, engineers are but little concerned with the first method of compacting and very greatly concerned with the second method. As the author's paper is largely taken up with a discussion of methods for insuring drainage by the lateral removal of water, the writer doubts whether it is as applicable to the solidification of the cores of hydraulic-fill dams as to the operation of sand filters.

Recently, at the Lockington Dam of the Miami Conservancy District, the hydraulic fill was interrupted and the surface water drained from the pool. As the surface of the core became dry, there appeared upon it hundreds of small springs discharging water which was being displaced from the core. In each case, a cone was formed of clay carried out with the water. These cones would become about 1 in. high and about 1 ft. in diameter. At another hydraulic-fill dam where work was discontinued until the water in the central pool became clear, these little craters were everywhere evident on the surface of the core under the water, with water pouring out of them like small springs. The confined water evidently forms vertical channels, and through these channels, rather than through the small interstices between the clay particles, the water escapes from the core. The character of these vertical channels can be understood by observing the markings which sometimes appear on the removal of forms from vertical concrete surfaces, especially where an excess of water has been used in mixing the concrete.

Mr. Morgan.

That similar vertical channels are formed throughout the mass of newly placed concrete where an excess of water is used, is evident from the appearance on the surface of similar craters from which water pours. Evidently, the size of cement particles does not determine the rate of the ejection of surplus water from the concrete.

Concerning hydraulic-fill dam construction Mr. Hazen states: "It has two fundamental advantages: First, in sorting out the fine particles and placing them in the center of the dam, thus insuring complete water-tightness; and second, in the use of power in place of muscle to the greatest extent." There are other material advantages of this type of construction. Work on a dry, rolled-fill dam commonly is interrupted during rainy weather. Work on a hydraulic-fill dam can continue almost regardless of rain and the condition of the surface. Where rapid progress or haste in making a closure is essential, continuous operation may become an important factor. A water-tight core is highly desirable in a dam, but so also are pervious toes. An ideal hydraulic-fill dam section is one in which the central portion is entirely impervious, whereas the outside portions are so porous that any leakage which may occur, will appear at the base of the dam where it can do no harm, rather than part way up the slope where it may cause a slip.

The author also states: "In general, all the material placed in hydraulic-fill dams may be divided into two classes, namely, toe material and core material." There are three fairly distinct classes of materials in a hydraulic-fill dam: First, the toe material which consists of coarse gravel and sand not readily transported by water; second, the core material consisting of extremely fine particles which remains in suspension, generally for days at a time, falling to the bottom only very slowly; and third, the transition zone between the core material and the toe material, which consists of fine sand, silt, and other material readily moved by water, but which will settle several feet in the course of an hour or so. It is distinctly different from either the core material or the toe material.

Mr. Hazen states concerning core materials: "As a practical proposition, drainage is not possible with such fine materials. By that is meant such prompt and complete drainage as will result in solidification during the construction of the dam." Nevertheless, the writer's experience indicates that consolidation and solidification do take place at a fairly rapid rate. At the Taylorsville Dam, the hydraulic fill was interrupted and a portion of the water drained away. The consolidation was so rapid that whereas at the time of stopping work an iron ball would sink 8 or 10 ft. in the core, within a few months it was possible to walk anywhere over the core, and a horse might even be driven over it. This is true both of that part of the core which was exposed to the air, and of that part which was permanently under water. About 40 or 50% of this core material is finer than the mini-

Mr. Morgan.

mum recommended by the author. A detailed analysis of this core material is presented in the discussion by Charles H. Paul, M. Am. Soc. C. E. When an auger hole was bored into the core, water would rise in the hole and flow out over the surface, being displaced by the weight of the superimposed core material.

Mr. Hazen gives favorable consideration to reducing the thickness of the core until it has almost disappeared, in this way eliminating much of the finest material. If the border line between the core material and the coarser material could be depended on to continue without change during the construction, this method might be feasible, although unnecessarily expensive, as a rule, since it would require the wasting of fine materials. There is no assurance, however, that such uniformity of core margins can be secured. If perfect workmanship and perfect control could be had during construction, one might be assured of an absolutely uniform core margin. As a matter of fact, the margin between coarse and fine material shifts within considerable limits. In the Conservancy District dams, a width of about 25 ft. is maintained on each side of the core where it is assumed that the condition is uncertain. In some cases, a tongue of toe material will shoot out beyond the theoretical lines and tend to make a gravel bar across the dam. In other cases, the flow of water into the pool will result in a bay or depression, leaving the core projecting into the toe material. With the greatest possible care, this fluctuation between toe material and core material can scarcely be held within closer limits than a belt perhaps 15 ft. wide on each side of the pool.

There may be cases where the cross-section of a hydraulic-fill dam might be reduced if lateral pressure of the core material could be eliminated. As a rule, there are other controlling conditions, and a dam properly designed from other points of view will be stable as far as pressure of core materials is concerned. In the Miami Conservancy dams the width of the core is held about equal to the distance down from the top of the dam. As these dams have steeper slopes at the top than at the bottom, the proportion of the width occupied by the core will vary. That the Calaveras Dam had too wide a core there is no doubt, and a definite limit of width of core is essential. In the design of the Conservancy dams, the possibility of failure during construction, due to excess width of core, was a prime consideration.

The experience of the engineers of the Miami Conservancy District leads to the conclusion that there are so many factors entering into the stability of a hydraulic-fill dam that it is not safe to single out any one or two of them and to rely on the theoretical determinations of those factors. The final conclusions are as apt to be empirical as theoretical; or, rather, many problems present themselves for solution, which never would be brought to the attention by theoretical deductions, but only by observation of construction in progress.

Mr. Morgan.

As the construction of the dams of the Miami Conservancy District nears completion, it is the intention of the engineers to make a careful report of their opinions based on this experience. As this construction work is scarcely half completed, it will be a year or more before these opinions and conclusions will be sufficiently definite for presentation.

Mr. Parsons.

H. DEB. PARSONS,* M. AM. SOC. C. E.—The author is to be congratulated on giving such a clear description of a most interesting subject. A rock-fill dam is not exactly in the category of an hydraulic-fill dam, because the material is not washed in by hydraulic streams, although such dams are made tight by the seepage of water through the rock fill itself. The speaker designed a rock-fill dam which was built in 1902, in the Ramapo Hills, near Ramsey, N. J., for the late Mr. George Crocker, of California.

Mr. Crocker desired a dam in order to secure a water supply for his estate, and he had planned a gravity-section stone dam, the plans for which he sent to the speaker for criticism and advice. After an examination of the site, the conclusion was reached that it would be a good opportunity to construct a rock-fill dam. There was rock adjacent to the site, and the earth had a certain quantity of clay mixed with it. The selection of a rock-fill dam was based on the expectation that its construction would be much cheaper and more suitable for the site than the original plan for a cut-stone gravity dam.

This dam is not to be compared, as far as size is concerned, with those which the author has mentioned. Although much smaller, the principles involved are true for dams of larger proportions. The dam was 230 ft. long on the crest, 20 ft. high, and had a top width of 15 ft. It was built with stone roughly dumped in the center, while on the down-stream and on the up-stream faces the stones were placed by hand, in order to obtain some regularity. The slope on the down-stream face was 1 on $1\frac{1}{2}$, and that on the up-stream face 1 on 1, but against the up-stream side an additional earth embankment was placed. The stone was laid roughly. The rocks were brought down from the hillside quarries, and the men simply dumped the core material in place, while carrying up the side faces a little ahead of the center core.

At the up-stream toe there was a cut-off, merely sheet-piling driven into the soil, with some parts ditched and puddled with clay. The pond above the dam covered between 11 and 12 acres. The site was prepared by removing the top surface soil so as to receive the stone which was brought down from both hillsides.

Two 36-in. pipes were built through the dam in order to control the water in the pond, and concrete was used around them to prevent

* New York City.

Mr. Parsons.

leakage of water and to give them strength. They were steel pipes, which, after a time, would rust out, and the concrete was intended to remain and take their place.

The dam had no spillway. It was not expected to be overtopped except at long intervals, and it was designed so that if it was overtopped, a flood would do no harm. The reason for paving the downstream side with large flat stones, especially picked out, was to prevent injury in case the dam should be overflowed.

The dam was finished in six months, having been started in May and finished in November, and its cost was $8 700, so that it is not difficult to figure the saving which this rock-fill construction made

This particular dam leaked during the process of construction, but at the end of six months, when it was finished, it was practically tight. Although it continued to leak, it was substantially tight thirty days after it was finished. Fine material had washed into the interstices of the rock that formed the core and had sealed the voids.

About twenty-five years ago, when the speaker was connected with the Nicaragua Canal Construction Company, the engineers proposed to construct the Ochoa Dam for the purpose of converting the Rio San Juan into a canal. This dam was designed to have an average height of 61 ft. above the river bed, a total length, including concrete abutments, of 1 900 ft., and a width at the base varying from 400 to 1 000 ft. It was proposed to construct the dam by depositing loose rocks and boulders which were expected to become water-tight by the lodgment of clay, sand, and drift. Although the dam received considerable discussion at the time, no question was raised as to its stability. There are locations where a rock-fill dam is more suitable than an hydraulic-fill dam.

Mr. Binckley.

GEORGE SYDNEY BINCKLEY,* M. AM. SOC. C. E.—The speaker is especially interested in Mr. Hazen's description of the Calaveras Dam and the causes which contributed to its disastrous failure. In this connection there are several points of importance to which it would seem pertinent to call attention. In the first place, where heavy material is deposited in the outer slopes of the structure by running water, the material is not only free from lubricant or light fines, but is incomparably better packed than when it is dumped from wagons or cars. This fact, together with the automatic segregation of coarse from fine materials, which should be characteristic of a well built hydraulic-fill bank, is something that should by no means be lost sight of when such a bank is compared with one constructed as a dry rock fill.

As the speaker pointed out in an article† soon after the failure of the lower Otay Dam some years ago, a dry rock fill of unsegregated

* Los Angeles, Cal.

† *Engineering Record*, June 10th, 1916, p. 769.

Mr. Binckley.

material is, if overtopped, capable of becoming gorged with water and in this condition its resistance to movement is enormously decreased, the hydrostatic pressure within the mass tending to its disruption, and its effective weight being decreased by the weight of water displaced. This condition as a matter of fact is what caused the destruction of the Lower Otay Dam, as the best of evidence exists to prove that erosion by the overpour had little or nothing to do with it. This example is mentioned for the purpose of emphasizing the difference in the character of slopes constructed by the hydraulic-fill process, and that of the outer slopes of the Calaveras Dam. Mr. Hazen states that during the earlier stages of construction, the pond, and, therefore, the puddle deposited, extended at times over the greater part of the width of the dam. This fact alone would be quite sufficient to account for the failure, for under such conditions a stratum of very plastic clay could exist between a base of coarser material and a superimposed mass through which the clay could not be extruded. If there is no outlet for the extrusion of a fluid or plastic material, it will support very great unit pressures while, at the same time, retaining its fluid or plastic character. Hence, if at any plane in the slopes of the dam there existed any such stratum of plastic clay, cause for surprise would lie, not in the failure, but in the endurance, of the structure.

Mr. Hazen mentions the possibility that under pressure the coefficient of friction might not be the same as under the conditions existing in the tests. It would seem to be quite safe to assume this possibility, for in the case of a material containing a more or less plastic clay in sufficient quantity to fill the voids, the unit resistance of the clay to change of form would be a constant, while the forces acting to produce movement along any given plane within the mass are variable. The conclusion seems inevitable that, under such conditions, the "coefficient of friction" (which in the case of the dam is actually a shear along the plane of greatest unit stress) would be higher where the forces in action are small, and rapidly decrease as such forces became greater in relation to the resistance of the plastic material to flowage or distortion.

The phenomenon of fluidity under motion of a material which is quite solid when it comes to rest, is mentioned by Mr. Hazen as having been observed in material which ran out from the Calaveras Dam. This is exactly the result to be expected if considered in the light of the foregoing remarks on the subject of the coefficient of friction. The same phenomenon was observed in the case of the Necaxa Dam. Great masses of material which flowed like water on a low gradient were found, when the motion ceased, to be solid, well compacted material containing relatively little water. More striking still are the great rock flows that may sometimes be seen in the West, a classic example being that of Turtle Mountain, in British Columbia, where an enormous

Mr. Binckley.

mass of rock slid off the top of the mountain, ran across a valley and well up the opposite slope before it came to rest—as solid a mass of rock and wreckage as if it had been built in place.

That part of the paper dealing with the Necaxa Dam has for the speaker a particular interest through his long association with the late James Dix Schuyler, M. Am. Soc. C. E., who designed this dam, and under whose direction it was built. In this connection, a little inside history relating to the conditions which preceded the accident to this structure may be of interest. Several months before the break in the up-stream slope occurred, Mr. Schuyler was showing the speaker the progress reports and cross-sections of the dam, and commenting on a curious condition which had developed—the rising of the core material at several points in the pond maintained above the core of the dam, to such an extent that islands were formed. This seemed somewhat of a puzzle, but after the speaker had suggested that it was caused by units of the heavy material of the slopes sinking into the core material and displacing it, he agreed that this was probably the explanation.

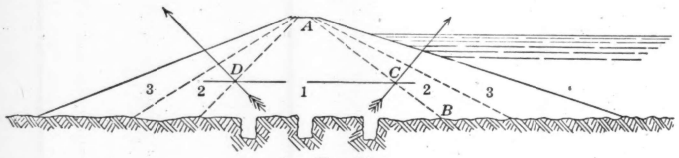


FIG. 21.

At this time, experiments had been made to determine the physical state of the core material, and it had been found that it was possible for two men, working from a boat in the pond, to force a 1½-in. pipe down into the core for a distance of more than 60 ft., demonstrating a condition of semi-fluidity or at least plasticity. This condition suggested to the speaker for the first time a danger that seemed not to have been recognized before, and he immediately called Mr. Schuyler's attention to it. Briefly, it would appear that where the mass of core material (placed in accordance with the designs as practically conventionalized by Mr. Schuyler) is in a semi-fluid or plastic state, it must be regarded as a heavy fluid filling the interior of the dam, and exerting a pressure against the interior of the slopes, not parallel with the base, but upward and normal to a plane corresponding to that zone within the mass at which the character of the material changed from plastic to non-plastic. Fig. 21 will make this clear. Zone 1 is assumed to be semi-fluid or plastic, and Zones 2 and 3 to be a non-plastic material.

It is plain from a consideration of this diagram that as long as the zone of plastic or core material is any wider at the bottom than

Mr. Binckley.

at the top, this condition as regards direction of pressure will exist in corresponding degree. It is also quite plain that the static pressure of the core material considered as a fluid, at Point *D* is dangerously near being in excess of the corresponding weight opposed by the overlying mass of slope material above Point *D*, even if this material is of a much higher specific gravity. Such a condition would threaten the disruption of the dam from internal pressure alone, quite aside from any question of sliding due to horizontal pressure. In the case of the Necaxa Dam, although the speaker believes that Mr. Schuyler was impressed with his suggestion, it was too late to make any change in the design of the dam, as the work was well advanced, and, as a matter of fact, there had been no alarming symptoms. Unfortunately, on account of the scarcity of suitable material, the placing of some very light rock in the up-stream slopes was subsequently permitted. This rock, as the speaker understands it, was practically the same as the well-known "caliche" of Mexico, and was very much lighter than the material previously used. This fact, coupled with the rapid draft of water for power purposes, which lowered the level of the reservoir and removed the support of the water from the upper slope, resulted in what may properly be described as the explosion of the dam. The internal pressure, in amount and direction, was too much for the resistance of the up-stream slope and it went out into the reservoir as did the up-stream slope of the Calaveras Dam.

Shortly after this accident the speaker met Mr. Schuyler in British Columbia, and the former conversation on the subject was recalled. He stated at that time that thereafter he would always limit the impervious core material to a relatively small cross-section, and if he had lived to construct other hydraulic-fill dams, there is not the slightest doubt that he would have corrected the error that he had come to recognize as existing in previous designs; and certainly it is hard to understand how, with the very plain lesson taught by the Necaxa case, the same error could have been committed in the Calaveras Dam, with the aggravating circumstances of slopes of dry fill and of tongues or strata of clay lubricant running out nearly to the slopes, as described by the author.

As regards the Coquitlam Dam, referred to by Mr. Hazen, the material in the core was of a very different character from that commonly found in hydraulic-fill construction. This material was a blue, glacial, so-called clay—actually a very fine silt, having rather remarkable qualities. It is a material with which the speaker became familiar a great many years ago while constructing a dam on the Puyallup River, and is the same as the bulk of material handled in the Seattle re-grade operations. Fine as this material is, and much water as it holds, it has the quality of setting very firmly in a short time. Indeed, its qualities are exactly opposite to those of quick-

Mr. Binckley.

sand, hence neither the speaker nor Mr. Schuyler had any hesitation in using it as a core material for the Coquitlam Dam. This work in collaboration with Mr. Schuyler, was designed and located by the speaker who conducted all the test-pit and tunnel explorations and the stripping of the site, etc. This structure was about 700 ft. long on the crest, was provided with a surface of heavy rip-rap laid in cement mortar both up and down stream, and was protected against overtopping by a spillway 15 ft. deep and, as the speaker remembers it, 250 ft. long through solid rock. For the greater part of its length, the dam rested on the same hard, tight, "glacial clay" already described, abutting on the east end against the diorite ridge in which the spillway was cut. As originally designed, this dam was perfectly safe, and any additions to its section were not improbably made in deference to the same element of the population of that district who opposed the construction of the dam under any conditions. It is possible, however, that a scarcity of suitable material for the heavy slopes developed during construction, in which case, of course, such increase of section as that actually given would have been wise.

In view of the costly lessons of failure and accessible experience in the accepted technique of construction, the desiderata for good design and safe construction of an hydraulic-fill dam would seem to be:

1. Plenty of coarse and heavy material in the slopes, deposited by flowing water, and segregated in such a manner as to form a progressively dense mass from the slopes inward toward the center.
2. The provision of adequate muck trenches or cut-off walls along the zone of the impervious core.
3. The careful avoidance of the introduction of any lubricant in the mass of the slopes.
4. The construction of an impervious core containing enough granular material to destroy or greatly reduce its plasticity, while, at the same time, containing enough clay to secure an impervious or substantially impervious core.
5. The provision of ample spillway capacity to guard against the possibility of the dam being overtopped, and in case of the slightest doubt, to provide a down-stream slope surface capable of carrying, without danger of erosion, a considerable overpour for a few hours.

Mr. Paul.

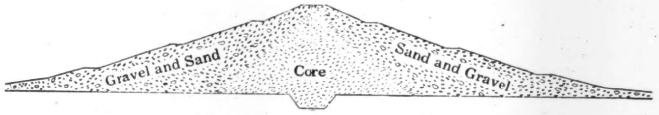
CHARLES H. PAUL,* M. AM. SOC. C. E. (by letter).—The writer is strongly in favor of comparatively narrow cores for hydraulic-fill dams. In several cases which have come to his notice, an unfavorable condition has been introduced by the fact that, in beginning construction, the core width at the bottom was allowed to be so great that the stable material in the outside slopes necessarily overhung portions of the core which had not yet consolidated. This is liable to result in sloughing of

* Dayton, Ohio.

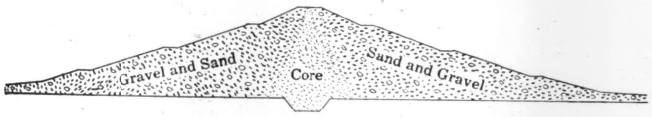
the coarser material into the core, thereby weakening the structure by permitting the core material to encroach still further into the region of the outside slopes. Mr. Paul.

In designing the dams for the Miami Conservancy District, the width of core at any point was limited to the height of the structure above that point. This gave a maximum width of core at the bottom of not more than one-fifth the bottom width of the dam, whereas, in many cases, hydraulic-fill dams have been built with core widths as great as one-third or one-half the bottom width of the dam. Comparison of the wide and narrow core is illustrated in Fig. 22.

One of these sketches, Fig. 22 (*a*), shows a core about one-third the width of the dam at the base, and it may be seen that before construction has progressed very far, the material in the outside slopes will actually overhang the core material. This is an undesirable condition. In the other sketch, Fig. 22 (*b*), the core width at any point is about



Type with Wide Core, Sides Subject to Falling into Core.
(*a*)



Type with Narrow Core, Sides of Superior Stability.
(*b*)

FIG. 22.

equal to the height of the dam above that point. There is very little overhanging of the coarse material, and under such conditions sloughing of the coarser material into the core is practically eliminated.

In a design with the core width at any point equal to the height of the dam above that point, there is no difficulty whatever in obtaining a cross-section stable against sliding and with side slopes as steep as will be permitted by considerations other than the lateral pressure of the core. This point also was carefully worked out in the designs of the Conservancy dams; as a matter of fact, the side slopes of those dams were determined by other considerations.

Fig. 23 is a view of Huffman Dam under construction, showing the comparative width of the core and the volume of stable material in the outside slopes. Dredge pumps located at the toe of the dam (at the stiff-leg derrick) handle the sand and gravel obtained from the

Mr. Paul.

river bottom, pumping it up into the embankment. The hydraulic giant in the foreground is knocking down the clay overburden on the hillside to supply the deficiency of core material. The concrete structure in the distance is a retaining wall forming one side of the outlet structure. The shape of this wall fits the final cross-section of the dam.

There is danger, however, of making the core too narrow, and the writer is inclined to believe that to follow Mr. Hazen's recommendation and make the core "as narrow as possible," or as illustrated in Fig. 3, would not be wise. It is not possible, in construction, to control the width of the core absolutely. Some leeway should be allowed, so that when the coarser material encroaches on the core for reasons beyond the control of the men on the job, there will be no danger of its shooting entirely across the core and resulting in a tongue of coarse material extending all the way through the dam. Also, there is a transition zone between the coarse material and the true core material, which must not be overlooked. The writer is of the opinion that a happy mean, between extremely wide and extremely narrow cores, is the proper thing, and he believes that the design adopted by the Miami Conservancy District is a conservative one from both viewpoints.

As to the gradation of particles in the core itself, several analyses have been made, from time to time, by the U. S. Bureau of Soils, at Washington, D. C., of samples taken from the cores of the five dams now under construction in the Miami Valley. Table 5 gives a typical analysis of the core material from each of these dams.

TABLE 5.—Typical Analyses of Core Samples.

	Bureau of Soils, standard sizing, in millimeters.	Germantown Dam. Percentage.	Englewood Dam. Percentage.	Lockington Dam. Percentage.	Taylorsville Dam. Percentage.	Huffman Dam. Percentage.
Fine gravel	2 to 1	0	0	0	0	0
Coarse sand	1 " 0.5	0	0.4	0	0	0
Medium sand	0.5 " 0.25	0	0.3	0	0.1	0
Fine sand	0.25 " 0.10	4.2	4.6	1.0	1.9	2.8
Very fine sand	0.10 " 0.05	17.3	19.6	10.1	18.1	15.0
Silt	0.05 " 0.005	56.0	52.7	59.6	59.7	52.5
Clay	0.005 " 0	22.5	22.6	29.3	20.2	29.8

It will be noted that although there is some difference in the gradation, yet, taken as a whole, the samples are after all remarkably similar. This in spite of the fact that each of the dams is in a different river valley, two of them (the Germantown Dam on Twin Creek, and the Lockington Dam on Loramie Creek) being 50 miles apart. A sample of Calaveras core, analyzed by the same methods gave very different results, as it was composed entirely of material finer than 0.05 mm., and 60.1% of it was finer than 0.005 mm. Undoubtedly, a

Fig. 23.—View of Huffman Dam Under Construction.

core of such material would behave quite differently from those represented by the analyses given in Table 5. Mr. Paul.

At the Taylorsville Dam there is an excess of fines in the material available for hydraulic fill, and a considerable percentage of the fines in the core are being wasted. On the other hand, at the Germantown and Huffman Dams, the borrow-pit material is deficient in fines, and additional fines are being sluiced in from the glacial till on the hillsides, to supplement the gravel and sand from the river bottoms. It is a source of satisfaction to the engineers concerned that, in spite of these necessary manipulations of the raw material, it is possible to secure the comparatively uniform results shown in Table 5.

Serious construction difficulties would be encountered at all the Conservancy dams if the limit of fineness of core material was to be held to 0.01 mm., as suggested by Mr. Hazen. Moreover, the cost of construction would be greatly increased, and presumably the imperviousness of the core would be considerably lessened. It should be said, however, that care is being taken not to have a large percentage of the very finest grade, namely, material finer than 0.005 mm. Some of this, it is believed, is quite desirable.

Satisfactory consolidation in the cores of all these dams is taking place, regardless of the fact that about 40 to 50% of the core material is finer than 0.01 mm. This is evidenced by rod tests, ball tests, and pressure-cell readings, as will be shown later.

The writer sees no great advantage in immediate consolidation of the core. Considerable time may be allowed for this process, because of the fact that during construction the height of the dam, and, therefore, the pressure from the partly consolidated core, does not increase greatly from month to month. Provided a reasonable rate of consolidation is assured, the writer feels that there is no need of going to extremes, or sacrificing water-tightness, to secure a more rapid consolidation.

The procedure being followed to secure satisfactory cores in the Conservancy dams and the methods of studying action of the material in the cores have been described in an article entitled, "Study of Pressures in Hydraulic Dam Cores."* This article describes in detail the Goldbeck pressure cells, several of which are in use in the dams of the Miami Conservancy District, for determining pressures in the cores as construction progresses, and particularly for determining the relation between vertical and lateral pressures at various elevations. About three years ago, the writer had occasion to make an investigation of the operation of the Goldbeck pressure cells under various conditions, and outlined with Mr. Goldbeck a series of tests which were conducted by the U. S. Office of Public Roads for the purpose of showing whether or not these pressure cells would give reasonably correct results if

* *Engineering News-Record*, December 25th, 1919.

Mr. Paul.

installed in hydraulic-fill dam cores. Rather elaborate experiments were made, which have been referred to elsewhere.*

The writer is satisfied that this apparatus is the best instrument that has yet been devised for determining the rate of consolidation and for making pressure studies under such conditions. Inasmuch as a full description of this pressure cell has been given by Mr. Goldbeck (page 1764), and also in the *Engineering News-Record* articles previously mentioned, the writer will not take the space here to describe the construction of this cell in detail. Very briefly, it is a closed flat circular box, 5½ in. in diameter (similar to a large shoe-polish box, in shape), the top or bottom of which acts as a movable diaphragm, operated by admitting compressed air, through a small pipe, to the inside of the box. Air is supplied from a small tank which may be charged with an ordinary tire pump. The box being buried in the material to be tested, the movable diaphragm is held down (or in) by the pressure of the material against it. When the pressure of the air admitted to the inside equals that of the material outside, the diaphragm is lifted slightly, an electric contact is broken, and an indicator light in the registering apparatus goes out. The pressure gauge on the air line, at that instant, indicates the pressure of the material against the movable diaphragm. The operation of the cell in taking such readings requires a movement of the diaphragm of less than $\frac{1}{10\,000}$ in. It works so positively that, by proper manipulation of the air feed, the light may be turned on and off any number of times without moving the pointer on the pressure gauge. The cell may be set horizontally, vertically, or at any angle, and thus may record the pressure from any direction. In this manner, valuable comparisons between vertical and lateral pressures may be secured.

There is no comparison between the operation of this pressure cell and the crude foundation tests referred to by the author, and even if they were comparable, the recorded pressures of the Goldbeck cells would be higher than the actual pressures, rather than lower, because of the principles of operation just described. Whether the actual readings are absolutely correct is not vital; the comparative readings are of great interest and value, and, to those who have used the apparatus, give promise of yielding information of considerable importance.

It would be a mistake to attempt to anticipate the conclusions which will finally be drawn from the complete record of pressure-cell tests and other core studies in the Conservancy dams. This subject was investigated very thoroughly before the construction of hydraulic-fill dams in the Miami Valley was undertaken, and extensive studies of the behavior of the core material in these dams have been carried on during the two years or more since construction began. Pressure cells will be installed at all the dams in their maximum sections. This in-

* *Engineering News-Record*, April 18th, 1918, and December 25th, 1919.

stallation at one of the dams has been carried to the extent where interesting results are already available. However, the data will be so much more complete after another year or two that it seems premature to attempt to draw final conclusions from those now at hand. With that limitation, however, the writer feels that the results obtained up to this time are worth presenting. Mr. Paul.

At the Germantown Dam, construction has been carried to a point 60 ft. above river level. This dam will have a maximum height of 110 ft. Pressure cells have been installed in the core at 10-ft. vertical intervals for determining lateral pressures, and at 20-ft. intervals for recording vertical pressures. The results obtained to date are shown in Fig. 24.

There is every indication from these results that the core at the lower elevations is consolidating in a satisfactory manner. The lateral pressures at the lower elevations are considerably less than the vertical pressures, and the diagrams indicate that progressive consolidation is taking place from month to month. At the present time, in the lower portion of the core, the lateral pressures are only about two-thirds of the vertical pressures. At the upper elevations, vertical and lateral pressures are about equal, confirming the generally accepted belief that, at these upper elevations, the core is still a perfect fluid. Viewed as a whole, the readings obtained so far at the Germantown Dam appear to be reasonably correct and suggest no reason for doubting that the pressure cells are giving approximately reliable data. There is also every indication that the core is consolidating rapidly enough for ample safety in construction. Similar data in complete form from all the dams will be available at a later date.

At the Taylorsville Dam, it has been possible to secure actual samples of the core at considerable depth from the surface, after pumping on that section of the dam had been discontinued for the winter. These samples, although they contain perhaps 50% of material finer than 0.01 mm., show without possibility of doubt that the core is solidifying at a satisfactory rate. In fact, at the present time at Taylorsville, it is possible to walk across the core without difficulty, and it has been decided before resuming work on this section of the dam to plow up the core, in order to be sure of obtaining a good bond for the new work.

Penetration tests with the 6-in. cast-iron ball are being made regularly at all the Conservancy dams. Fig. 25 shows the record of ball tests at the Germantown Dam.

The ball sinks to a maximum depth of only about 16 ft., while at the Calaveras Dam the writer is informed that in many cases penetration of the 6-in. cast-iron ball was as great as 50 to 60 ft. Several tests have been made by leaving the ball in place for a period of several days in order to see whether it would gradually continue to settle,

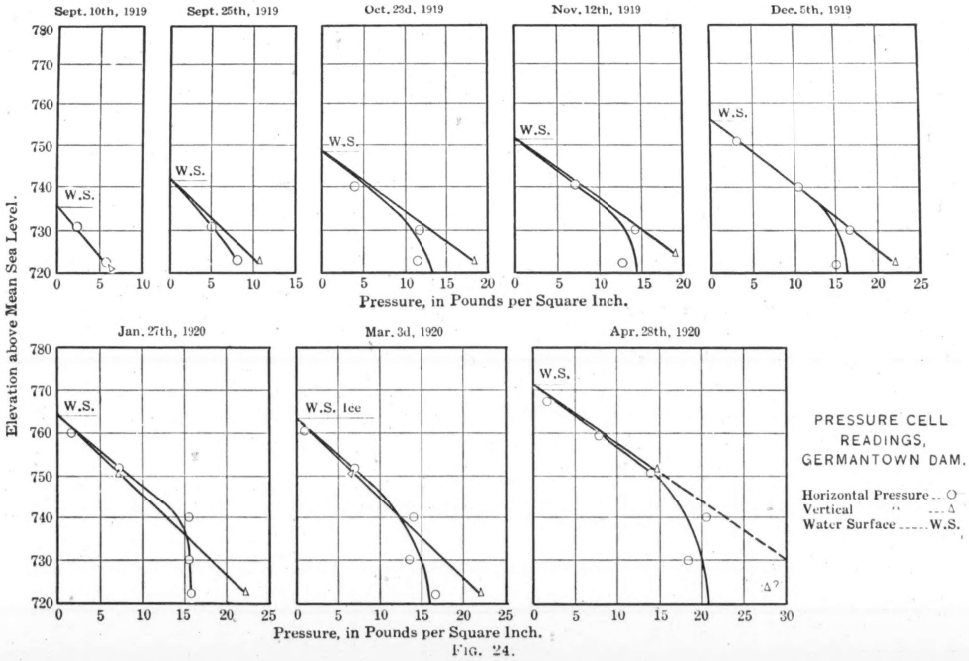


FIG. 24.

Mr. Paul.

Mr. Paul.

PENETRATION TESTS—6 INCH CAST IRON BALL
GERMANTOWN DAM.

Fig. 25.

Mr. Paul. but it has been found that after the first minute the amount of settlement is generally negligible. It is true that material which will support a 6-in. cast-iron ball is not necessarily very stable, but, on the other hand, it is definitely along the road toward consolidation. It can hardly be called a fluid. Moreover, these ball tests are valuable, owing to the fact that they are being used in other hydraulic-fill dams, and therefore, give means of comparison at dams in various parts of the country. The great difference in ball penetration at Calaveras and at the Conservancy dams, is significant.

Rod tests have also been made at the dams in the Miami Valley, but little value is attached to the results obtained by these methods, as the personal element has such a marked effect on the results. There is nothing very definite about a penetration obtained by two men, working from a boat, with an iron pipe or rod 30 or 40 ft. long. These rod tests, however, do confirm the indications shown by the pressure-cell tests that a satisfactory consolidation of the cores at the lower elevations is taking place. The writer's experience does not confirm the author's statement that material too stiff to be penetrated by the rod will still exert full lateral pressure. This may be disproved by working the rod test in a natural clay bank.

It is interesting to note in this connection, also, that maximum rod penetrations in the Conservancy cores do not exceed about 30 ft., while, at Calaveras, the rod penetrations are reported to have reached depths of from 80 or 90 ft.

The author states that material with 50% voids "is capable of standing up in a boring driven beyond the casing for several feet" and, in another paragraph, that this same material "exerts full lateral pressure, corresponding to its height and weight per cubic foot." The two statements appear to be inconsistent. There is a question in the writer's mind whether the percentage of voids in such material is necessarily a measure of stability. He doubts whether void determinations from boring or other samples are reliable, even as representing the true condition of the core itself, because it is so extremely difficult to bring up a sample in which the moisture content or conditions of compactness has not been disturbed. This is a point which is often disregarded, but which, nevertheless, is very important. The very act of taking a boring sample, especially in such material, tends to compact the sample and change its moisture content.

To one who has not made a study of the history of hydraulic-fill dams, the impression might be gained from Mr. Hazen's paper that many such dams have failed. Such is not the case. Aside from the two failures which have been mentioned, namely, Necaxa and Calaveras, both of which have been fully accounted for, the writer has been unable to learn of any other instances of serious difficulty with

dams built by approved hydraulic methods. On the other hand, there are many such dams in service, throughout the country, at which there has been no suggestion of trouble. In fact, it is apparent that, in a hydraulic-fill dam, construction conditions are generally more severe than those which will be encountered in service; a decided advantage for that type of dam. Mr. Paul.

JOHN E. FIELD,* M. AM. SOC. C. E. (by letter).—The writer was glad to see Mr. Hazen's paper, because heretofore the hydraulic method of construction for earth dams has seemed to be in disrepute, not only with the general public, but among engineers. This disrepute is the result of the failures of the Necaxa and the Calaveras Dams, in particular, and to some other failures not so well known. The failures have naturally attracted more attention than the successes, of which latter there are many. There are in Colorado alone three successes, namely, the Terrace, Santa Maria and Onion Valley Dams, and one failure, the Schaefer Dam, near Florence, Colo. In the case of the Schaefer Dam, the material was entirely unsuitable, being either of yellow clay, or rough angular sand rock, with very little sand or loam. The clay puddle remained the consistency of cream, and when the dam was finally finished by steam shovel and train, the central puddle material was allowed to rise as the dry fill progressed, and finally was drawn off. Mr. Field.

In considering the phenomena of engineering failures, there is generally a difference of opinion, and some mystery; especially is this true of the failure of earth dams. Doubtless, much valuable information and data are suppressed in cases where litigation over damages is in progress. The absence of trained observers before and during the time of failure, and the unreliability of testimony by the casual, but honest, observer, make these differences of opinion possible. In the case of the failure of the Calaveras Dam, the observations are unusually complete and reliable, and its discussion by the Profession should be thorough. This is the excuse for the random comments here presented.

In discussing hydraulic-fill dams, the very different conditions and objects to be attained must be kept in mind. Recently published articles on hydraulic-fill dams have considered particularly the construction period. The troubles at Necaxa and Calaveras were not properly dam failures, and to the general public were of little interest, as they involved neither its life nor property; the owner and the engineer were intensely interested and the latter has given considerable prominence to the phenomena presented. It is better to have one dam in ten fail during construction, than to incur the enormous expense necessary for all in order to insure against the one failure. On the other hand, the failure of one dam when in use involves such great and

* Denver, Colo.

Mr. Field. widespread consequences that any extra precaution and expense is justified to insure the safety of all.

In the failure of earth dams by slipping that have come to the writer's attention, one cause has seemed to predominate, namely, excessive width of core at the base in the ease of hydraulic fills, and poor foundations or permeable base in the case of other dams. The slips at the Wachusetts Dam and at Standley Lake near Denver are examples of the latter.

The writer has read the published accounts of the failure at the Calaveras Dam, and has talked with some engineers who visited the dam before and after such failure. The cause seems to the writer to be quite certain, that is, a layer of fluid material existed under a portion of the upper toe. Whether this layer extended to, and connected with, the puddle core proper is not important. The weight of the dry fill or of the core, or both, transmitted pressure through the layer of fluid material to the lower portion of the upper slope, probably finding the weakest point at or near the upper edge of the concrete paving, for Mr. Hazen has made the following statements:*

"The lower part of the toe had been paved with concrete and this extended 13 ft. above the water level. The top of the concrete formed a conspicuous line and observers say this seemed to rise several feet. * * * * saw the tower 230 ft. high swaying back and forth for a moment, after which it fell in a direction straight away from the dam. * * * * The concrete slope (June 18th, 1917) lifted and cracked parallel to the upper edge and 6 ft. below it for a distance of 700 ft., more or less across the dam."

It appears that a lift of the face near the upper edge of the concrete preceded or was simultaneous with the horizontal movement; whether there was an upward movement of the surface of the fill below the water line is not known, and it may be that the water pressure threw the minimum effective weight of the toe just above the water line, and perhaps also the water caused the puddle on the upper toe in the base of the dam to remain more fluid and to drain less readily than the lower toe which, although lighter in section, did not fail, and the water also saturated and rendered less stable the supporting upper toe. The fact that so many slips in dam enbankments have been on the upper slope, and at a time when water was in the reservoir, leads the writer to suspect that that condition is a contributing cause to failures. The objects of letting water rise on the fill are two: first, to help support the upper toe, and, second, to make use of the water so stored. If the water is drawn off for use, then the first object is lost. In fact, the drawing down of the water may be a menace, as it would leave a more or less saturated upper slope of less stability to support the core pressures.

* *Engineering News-Record*, April 4th, 1919.

Mr. Field.

This phenomenon is most marked in caving river banks when floods begin to recede.

It is to be remarked that the failures both at Calaveras and Necaxa were toward the flatter slope, up stream. In the first, there was water in the reservoir, while in the latter there was none at the time of failure, but there had previously been water against the upper face.

It is only natural that in the process of construction the puddle material should extend farther up stream from the center line under the flatter slope than under the steeper lower slope, unless such a result is guarded against. The engineer building any dam, naturally and properly has in mind the safety of the work when in use, rather than its safety during construction. For the completed hydraulic fill, he strives for a well drained lower toe of as great cross-section as is practicable. He also strives for an impervious core of ample thickness, but for the upper toe he desires only a protective coating and support for the puddle core. In ordinary fills the farther up stream the impervious core is, the better. All these considerations tend to place the puddle up stream and under the flatter upper face, and this, if carried too far, would result in failure during construction.

In construction, it seems wise to predetermine the position and minimum thickness of the puddle core and to give the greater stability to the upper toe, especially where water is permitted to rise in the reservoir. The presence of excess width of puddle material in the lower portions of the Calaveras Dam would account not only for the failure, but for the discrepancy in the coefficients of friction as determined by experiment at San Pablo, and as indicated by the failure at Calaveras. The planes of rupture formed by settlements prior to the strengthening of the upper toe were not eliminated by such strengthening, and remained a source of weakness not mentioned by the author.

Mr. Hazen states, referring to hydraulic fills, "experience does not show that the material is graded according to the theoretical section." The coarse materials are deposited near the toes and then filled with finer materials, making a very heavy compact mass with a low percentage of voids. In the experience of the writer, toes so placed were sufficiently impervious during construction to prevent appreciable seepage through them. In practice, also, the sand settles immediately on reaching the water of the pool, and rapidly pushes toward the center, and with each successive raising, and the occasional lowering of the water in the pool, the effect is somewhat as shown in Fig. 26. The points of sand extending into the puddle aid in the drainage.

The minimum thickness of the puddle is the minimum distance between points of sand touching the area, *A*, of the diagram, and the maximum thickness of the puddle is the maximum width of puddle within areas, *A*, *B*, and *C*. In calculating the weights of toes, the

Mr. Field. minimum thickness of core should be taken, while if the sand and puddle are incorporated by artificial mixing, the thickness of the impervious diaphragm may be taken as the maximum width of the puddle.

In connection with the drainage and saturation in hydraulic and ordinary dams, it may be of interest for the writer to state that at the Terrace Reservoir he had occasion to excavate into the puddle core and also into the portion of the dam placed by teams. The lower 170 ft. of the dam was hydraulicked, while the upper 30 ft. was placed by teams and wagons and thoroughly rolled. In the excavation mentioned, the puddle core was nearly as stiff as material used for making stiff mud brick, and the consistency of the team fill was the same, as far as could be determined by handling and appearance, showing that the team fill had taken on moisture, and the hydraulic fill had lost moisture, until they had each apparently reached a common degree of saturation. Within the line of saturation, one should expect to find similar degrees of saturation in either class of dam.

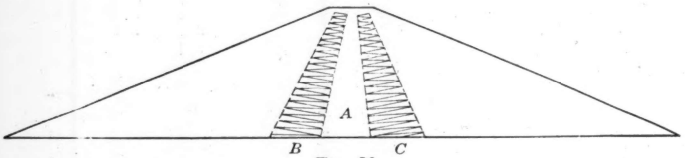


FIG. 26.

At the Terrace Reservoir, the water used in sluicing, particularly during low water in the river, contained a very appreciable quantity of alum in solution, so that it was perceptible to the taste, and its use for domestic purposes was avoided. This fact may have had something to do with the very satisfactory settling of the core material. While fluidity in the puddle increases with depth, increased depth also aids drainage, for under the high pressure present in high dams the water is forced out laterally and the material is consolidated. In the hydraulic fill, the pressure is gradually increased, and the drainage and consolidation are completed in the lowest layers first, the upper layers adjusting themselves as the shrinkage, due to loss of water, progresses. In this matter, the experiments and tests being made by the engineers of the Miami Conservancy District are enlightening.

The author says:

"If drainage and consolidation of the core are not secured, however, safety can only be reached by making each toe large enough to stand the full pressure of the liquid core, and, in that case, in order to be safe, a dam built by the hydraulic method must have flatter slopes and a larger section than is otherwise needed for an earth dam."

This statement is obviously true if one grants the premises for the hydraulic fill, and considers only the dimensions necessary to resist

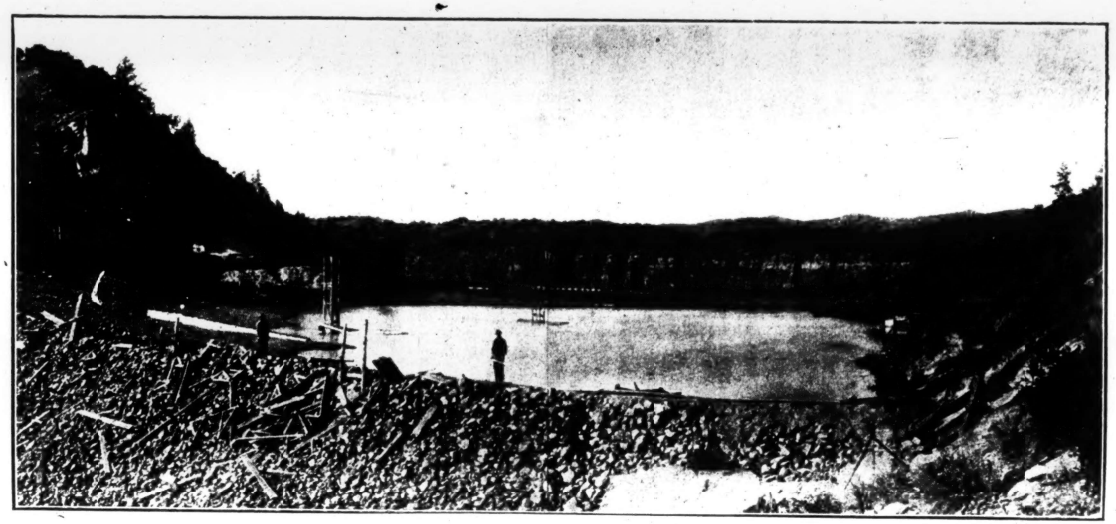


FIG. 27.—TOP OF TERRACE DAM, LOOKING NORTH (UP STREAM), SEPTEMBER, 1910, SHOWING CHARACTER OF TOE MATERIAL, WASTE WELLS AND SETTLING BASIN.

sliding and percolation for the dry fill. In earth-fill dams, however, the slopes, in order to stand, must be at least 1½ to 1, but in order to keep the lower slope outside the line of saturation, it may have to be increased materially, and to have a firm stable bed for the rip-rap facing in the upper slope, it too must be increased. With loose rock rip-rap, a slope of 2½ to 1 is none too flat, and a slope of 2 to 1 for the lower slope is necessary, as 1½ to 1 slopes are not stable under the action of the elements. We may assume, in most cases, that the minimum section of an earth dam is a base five times the height plus the crest width, and it remains only to consider whether such a section would be sufficient for an hydraulic fill. Mr. Field.

The writer wishes to comment on that portion of the paragraph quoted, which reads "If drainage and consolidation of the core are not secured, * * *". It is his opinion, first, that with the material generally found, the core does drain and consolidate, and will be self-supporting, and second, where puddle material is not satisfactory, that drainage and consolidation can be secured. Narrowing the settling basin and wasting the material, as suggested, should be avoided where material is as scarce and difficult to obtain as at the Terrace Dam in Southern Colorado. The scattering of coarse material over the surface of the settling basin would aid in both the drainage and consolidation of the core. The core material should be homogeneous, but need not be uniform in size. The ideal condition would be a mixture giving the greatest weight per unit, the greatest impermeability, and the most stable character. Such a material would be a mixture of coarse and fines, as in concrete aggregate. The writer believes that the presence of large boulders in the core is no detriment; that in time such boulders placed in the core would settle to a point where the core would be quite stable. Mr. Hazen states that "material with 50% of voids in a considerable depth, acts essentially as a liquid." The settling of boulders through and into such material would probably entirely overcome the increasing instability due to depth.

However, in practice there should be added to unstable core material, and thoroughly incorporated therein, boulders, gravel, and sand. The method of adding these coarser materials need not be expensive, and will vary according to the plant, equipment, and local conditions. If the minimum thickness of core is first determined, and then measures are taken to secure stability, there need be no material widening of the base beyond that required for other methods.

The author states that "If a factor of safety of 2 is required, the outside slope must be 4 to 1" [on the basis of a coefficient of friction of 0.5 and equal weight of core and toe material] "but in view of the records of slips in actual dams, it may be well to take a higher factor of safety." With this conclusion the writer cannot agree, for the following reasons:

Mr. Field.

First.—As the coefficient of safety is to apply only to the period of construction, a factor of safety of 2, or even less, may be assumed, as the consequences of failure do not involve the public safety, but a factor of 3 would involve very greatly increased expense. At Necaxa, with slopes of 4 to 1, the volume in the fill would have been doubled for the lower toe, and 33% greater for the upper toe, or the 2 000 000 cu. yd. would have been increased far more than was lost by the slip. The same is true of the Calaveras failure, but even on slopes of 4 to 1 and assuming too wide a puddle at the lower levels, there is no assurance that the failures would not have occurred.

Second.—The lower and lighter toes at both Necaxa and Calaveras did not slip.

Third.—The weight of material in toe and core is not equal, but the two materials as suggested by Mr. Hazen are generally as 5 to 4. Assuming that the toe material has a weight of 120 lb. per cu. ft., and the puddle material 100 lb. per cu. ft., with slopes of $2\frac{1}{2}$ to 1, and a coefficient of friction of 0.5, the factor of safety would be 1.5, which seems to be sufficient for the construction period.

Fourth.—Because defects in execution are revealed in the records of the failures, and may now be avoided.

Fifth.—In the increasing of the factor of safety, Mr. Hazen increases the slopes, but increasing the slopes has a tendency to widen the puddle core and extend it under the slopes, which seems to have been the cause of failure at Calaveras.

The writer accepts generally the principles enunciated by Mr. Hazen, but arrives at somewhat different conclusions, as follows:

1.—To limit the size of material to 0.01 mm., about $\frac{1}{2\,500}$ in., would not necessitate wasting more material than would naturally be wasted in the process of construction, under ordinary conditions.

2.—The puddle should be at a minimum, especially in the lower portion, and care should be taken to have the toes of full width, particularly at the base of the dam.

3.—The study of borings and the measurement of vertical and horizontal pressures, as on the dams being built by the Miami Conservancy District,* are essential.

4.—An ample factor of safety during construction seems to be 2, as the added volume due to a greater factor of safety would generally be greater than the probable loss should failure occur.

5.—Insure the solidity of toes by the use of rock fill placed hydraulically or otherwise. If placed hydraulically, greater weight will generally result.

6.—It is, in the writer's opinion, unnecessary to provide against quicksand conditions.

* *Engineering-Contracting*, June 16th, 1920.

Mr. Field.

A strict application of the principles laid down would probably not reduce the number of hydraulic dams, since their observance would increase the confidence of the public and the Profession in them, although a few, where clay was practically the only available core material, would not be undertaken. Observance of these principles would not materially increase either the volume or cost, but with proper testing would eliminate present uncertainties, and result in the more extensive use of this most useful and economical method of dam construction.

Mr. Hazen states that a 6% slope was necessary at the San Pablo Dam for sluice flumes with wooden bottoms. At the Terrace Reservoir the flume had a grade of 2.9 ft. per 100, and was lined with 6-in. wood blocks, much rock weighing from 50 to 100 lb. was carried, and some was as heavy as 200 lb. The rock was in part rounded, and in part angular; these flumes gave as good carrying efficiency as those lined with steel on the same grade, and were much more economical.

It is to be hoped that further investigations and experiments will be made to determine whether the suggested minimum size of 0.01 mm. for puddle material cannot be very materially increased or decreased with safety; whether the core material, even though of clay, does not drain laterally and consolidate so as to become self-supporting; and whether it is wise to allow water to rise in the reservoir as the fill progresses.

Mr. Holmes.

J. Albert Holmes,* M. Am. Soc. C. E. (by letter).—In this exceedingly interesting and valuable paper, Mr. Hazen has demonstrated the importance of a narrow central pool in hydraulic-fill dam construction and the influence a narrow pool is likely to have on the grain size of the core material.

As a result of his study, he proposes as a minimum for core material an effective size of 0.01 mm., and to waste from the pool any particles finer than this size. The writer is greatly interested in both these features and believes in their effectiveness in producing a stable and impervious structure. He has, however, brought about these conditions in a somewhat different manner.

In Table 2, Mr. Hazen has listed two dams, with one of which the writer was connected as Resident Engineer, and with the other as Resident Engineer for the Consulting Engineers.

The Somerset Dam, at Somerset, Vt., is in the region of the northern ice fields, and the material of construction is a boulder clay. The Paddy Creek Dam is one of a group of four dams at Bridgewater, N. C., and is built of the decayed and disintegrated rock of a non-glacial country.

* Decatur, Ill.

Mr. Holmes.

The method of construction was semi-hydraulic, that is, the material was excavated by steam shovel, loaded into dump cars, hauled to the dam site by steam locomotives, dumped from trestles and dikes, and sluiced into a central pool where segregation and sedimentation took place.

At Somerset, the sluicing water was pumped from the reservoir and delivered through pipe lines laid along both dikes, with hose connections at frequent intervals. The previously loosened material in the dikes did not, of course, require the same quantity or velocity of water necessary to excavate hydraulically and transport material to a straight hydraulic-fill dam. The quantity of water used at Somerset was about 1 sec-ft. as against 15 to 20 sec-ft. at San Pablo.

To regulate the height of pool at Somerset, an overflow was provided. During the first stages of the work there was a discharge from the pool, the flow carrying with it an estimated quantity of 100 cu. yd. of fine material per 24 hours. It was sought to retain this material, rather than to get rid of it, and during much of the time there was no overflow from the pool and the fine particles were retained. That this could be safely done was due to the character of the borrow-pit material, the narrow pool and core, and the wide supporting dikes. Before the pool was formed, the outer dikes were built up by dumping, first from the trestles and then over the edge of the dikes.

At Somerset, it was desired to give the core a broad base, covering the middle-third of the dam and making seepage travel long at the junction of the core with the stripped surface. There was no cut-off trench. After forming the broad base the core was narrowed quickly so that at completion it occupied one-fifth the cross-section.

The pool width varied from about 60% of the width at the base of the dam, to less than 30% at the top. Sluicing from the dikes into the pool left all gravel and boulders in the dikes. The size of the rock going into the dam was limited only by the capacity of the shovels to load it.

The dikes were watched carefully for movement during construction, but no movement was detected. They were broad and stable at all stages of the work. A complete mechanical analysis was not made of the Somerset core, but individual samples showed 94% passing a 200-mesh sieve, the maximum size being about 0.10 mm.

The core of this dam would probably be placed by Mr. Hazen in the 0.001 to 0.002-mm. effective size class. The dam has been in use nearly seven years and is perhaps entitled to be listed with "A Number of Earth Dams That Have Stood Successfully."

The material of which the Bridgewater Dams were built contained very little that could be classified as clay, and the deeper the excavation was carried in the borrow-pits, the coarser the material became.

Mr. Holmes.

The problem here was to get enough fine material to form the cores. On the other hand, the disintegrated rock did not furnish sound and durable gravel and boulders to give weight and permeability to the outer dikes, although the resulting structures were in every way satisfactory.

The Somerset method of dumping from trestles and dikes was followed at Bridgewater. The sluicing was done from scows floated in the pools on which were mounted monitors and pumps, taking the water from the pools and using it over and over. No overflows were provided; the seepage and evaporation losses amounted to about 0.4 sec-ft. and were made up by pumping from the reservoir to the pool. By this method of sluicing, that is, with water being taken from and returned to the pools and without overflows, all the fine material was retained.

Seepage experiments were made with two combined samples, made up of a large number of smaller samples taken from all parts of the core of two of the dams. These experiments gave a seepage rate for one dam of 30 000 gal. per 24 hours and about 14 000 gal. per 24 hours for the other. This is considering only the core of the dam, from which the samples experimented on were taken.

Mechanical analysis was made of the cores of the three larger dams at Bridgewater. The analysis shows very similar material in the cores of all three dams and that the effective size is nearly three times that suggested by Mr. Hazen as a minimum.

TABLE 6.—EFFECTIVE SIZE AND UNIFORMITY COEFFICIENT OF CORE MATERIALS.

	Catawba.	Linville.	Paddy Creek.
Size at 60% by weight, in millimeters	0.146	0.150	0.157
+ Effective size (at 10% by weight), in millimeters	0.026	0.028	0.030
= Uniformity coefficient	5.6	5.4	5.2

The Bridgewater Dams have rock in the down-stream toe; at Somerset there is some rock in both toes. The Somerset method of dumping from trestles and dikes and sluicing the fine material from the dikes to the core, insures stable dikes and ready control of pool width and character of core material, and in the writer's opinion, permits the use of a fine core material.

Mr. Swanitz.

H. W. SWANITZ,* Assoc. M. AM. SOC. C. E. (by letter).—This paper on hydraulic-fill dams sets forth in detail certain principles and theories to be followed in constructing earth-fill dams by the hydraulic process. The author treats the problem as essentially one of filtration and cites

* Vallejo, Cal.

Mr. Swanitz.

an elaborate series of tests and experiments involving microscopic work to determine the size of the grains in the central impervious core, as well as certain tests to arrive at the coefficient of friction of the material.

The paper is based primarily on tests made as a result of the failure of the Calaveras Dam of the Spring Valley Water Company, in California. The assumption is made that practically the same laws of capillarity and drainage apply to the very fine-grained clays as to the fine-grained sands. This may or may not be so, but it has been well known for several years among certain members of the Profession who have had to deal with earth dams, that fine-grained clay drains very slowly and that any embankment (as the core-wall of a dam) deposited under water, will remain in a supersaturated condition for a long time, and, moreover, that this material exerts pressure comparable to pure hydraulic pressure due to a fluid of increased density, the density being that of the core mass.

In the construction of any dam, the type of structure, as well as the method to be used in its construction, is usually indicated quite clearly by the following: (1) the materials at or near the dam site; (2) the importance of the function of the dam (that is, whether it is to form storage for domestic water supply, or power development, or flood protection, etc.); and (3) the amount of money which may justifiably be spent in building the dam. With these facts in mind, if an earth dam is the correct type, then the method of constructing it should receive careful attention. From the list of failures of earth dams, and the well known fact regarding hydraulic pressure just mentioned, it would seem that in the case of any important dam of considerable height, which is to be built in a locality where the available material is mostly pure clay, or rocks largely of a clayey nature, the hydraulic method of construction is definitely indicated.

The whole subject of the design and construction of earth dams reverts to the fundamental fact that the engineers in charge must have a thorough understanding of the nature of the materials which are to be used in the dam, and appreciate the practical limitations of these materials when used in large masses, either alone or mixed with each other in various proportions. This knowledge should be obtained and applied before the dam is built.

A hydraulic-fill dam should be composed of materials which, when finally in place in the structure, will be practically water-tight throughout a portion of the cross-section. If this water-tight portion is to be confined to the central part of the dam, it is believed that it will be a distinct advantage to have the up-stream prism of the dam as compact and nearly water-tight as is reasonably possible. This prism will then be more stable in itself and, due to its added weight and solidity, will

Mr. Swanitz.

offer greater resistance to sliding and settlement both during construction and afterward. The prism of the dam on the down-stream side of the core-wall should be of a comparatively porous nature in order to allow an easy means of escape for the water which drains horizontally from the core during construction, and also for any water which may appear later as a more or less permanent seepage through the dam. This down-stream prism plays an important part in the ultimate resistance of the dam, both against sliding and also in case the structure should be overtopped at any time during construction, or later. For these reasons it will be found advantageous to use a rock-fill toe of more or less generous proportions, as well as a heavy covering of a large rock rip-rap over the entire down-stream face. In important structures an ample freeboard should also be provided.

The dam must have slopes sufficiently flat to insure the stability of the structure under all conditions of saturation from the water in the reservoir. In most cases a slope of 3 to 1 up stream and 2 to 1 down stream will insure this, if the correct materials are used and if they are correctly placed.

The spillway of an earth dam must receive careful consideration as to its capacity, location, and method of construction, as it is the final safeguard against destruction in case of an unprecedented flood occurring with a full reservoir. The dam should be built so that no large cracks will form and no undue settlement will occur after completion. Core material must be of such a nature and placed so that it will drain and solidify while construction is in progress, at least to a point where it will not readily transmit full pressure as a hydraulic medium from the core material at a higher elevation.

These conditions regarding the core clearly indicate that one made of pure clay in which much of the material is very fine-grained should not be used, as it will undoubtedly form a mass of supersaturated liquid mud if placed at the ordinary rate of hydraulic-fill construction. This occurred at the Calaveras Dam. The core should be composed of a mixture of clay, sand, and gravel, the percentage of clay being just sufficient to fill the voids. Such a core will become very stiff and resistant on drying out; moreover, it will drain itself and solidify sufficiently during construction to prevent the transmission of hydraulic pressure or undue settlement in the finished structure.

Mr. Hazen points out that more rapid and thorough drainage and consolidation of the core material will be obtained by wasting the finest particles of clay, and the writer agrees with this, especially where the core material consists of an excess of clayey matter. This in effect amounts to abolishing the method of maintaining a comparatively still, standing, "summit pool", which allows of nearly complete deposition of all hydraulicked material. This fact has been appreciated and used

Mr. Swanitz. by some engineers in hydraulic-fill construction for a number of years, notably in the regrading at Seattle, Wash., and other work.

In cases where considerable doubt exists as to the correct method of building an earth dam (especially one of great height, with an important function to perform) the dry-fill method should be adopted without hesitation. This is certainly the case at Calaveras.

The tests and experiments given by Mr. Hazen would no doubt be of some value in furnishing a rough preliminary idea of the stability that might be expected of a structure built with certain materials in various combinations. However, it is not believed that in the majority of cases it is necessary to go to this trouble.

Mr. Justin. JOEL D. JUSTIN,* M. AM. SOC. C. E. (by letter).—This paper proves conclusively what many engineers had before only dimly realized, namely, that the inclusion in hydraulic-fill dams of the finer particles of certain clays, introduces an element of danger which should be eliminated either by the exclusion of such minutely fine particles or else by the addition of especially heavy toe fills of rock or other stable material to counteract the semi-liquid pressure of the core.

It appears to be quite generally recognized that the use of a central pool is an essential feature of the process of constructing an earth dam by the hydraulic sluicing method. Of course, a large central pool with a long period of settling is an essential feature if it is considered necessary to deposit from the water all or nearly all the suspended particles. Mr. Hazen, however, has shown that this is undesirable, as it leads to an unstable condition of the core. He advocates a great reduction in the dimensions of the central pool from those in current use, so that the period of settling will be shortened and the finest material not deposited.

The writer, however, raises the question: Why use a central pool at all? The fine impervious material will do the most good at the up-stream face and may be placed there by keeping the sluicing trestles at or near the down-stream face and allowing the soil-laden water to be discharged in an up-stream direction. Fig. 28, which is diagrammatic merely, indicates the various stages in the construction of an earth dam by this modified hydraulic-fill method. The material at the down-stream face is kept heaped up so that no water flows over it. As the water is discharged from the trestle, a great deal of the coarser material will be deposited in the vicinity of the trestle.

The slope is built up so that the finer material, with the water carrying it, will flow up stream. At the up-stream face a low brush dam is maintained, such that there is a small pool near the up-stream face, say, 1 ft. or 2 ft. deep and 15 ft. wide. The surplus water bearing the finest material is allowed to flow over the brush dam and down the up-

* Winston-Salem, N. C.

stream face. If plenty of brush is used, this practice will not cause much trouble from erosion, as the brush protects the face quite effectively. If brush is not available, baffle-boards may be used, and the surplus water and the rejected fines disposed through sluices or pipes. Mr. Justin.

The condition then existing is quite analogous to that illustrated by Mr. Hazen in Fig. 10, where the central pool was contracted to very slight dimensions, resulting in the deposit of material having an effective size of 0.01 mm., which material soon became stable.

With the small pool mentioned, located at the up-stream face, the material deposited would soon become stable. In fact, it would be entirely impossible to use a large pool and deposit a fine material which did not soon become stable, for a brush dam or other slight barrier at the up-stream face would not be strong enough to retain such a semi-liquid fill which would at once break out and flow down the up-stream face.

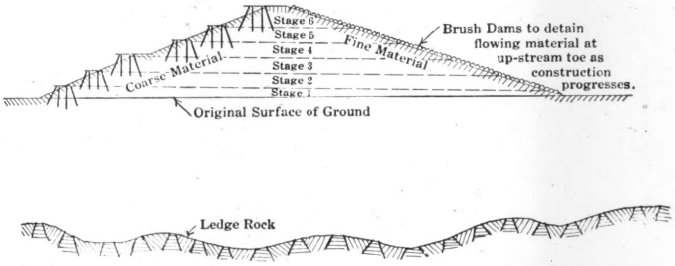


FIG. 28.—METHOD OF CONSTRUCTING EARTH DAM BY SLUICING WITHOUT THE USE OF A CENTRAL POOL.

A modification of this method would be its use in connection with a dam constructed by the semi-hydraulic-fill process. In this method, trains of dump cars are run out on the trestles and dumped. After the cars are dumped, small giants are turned on the dump under the trestle and the material is sluiced away in an up-stream direction, the heavier rocks remaining in the position in which they are dumped from the cars. The remainder of the method is the same as that already outlined.

By excluding the finest particles from the dam, a structure is obtained which, although not theoretically as impervious as if these excessively fine particles were included in the fill, is, nevertheless, practically water-tight. It is claimed for this method that it places the finer material near the up-stream face where it will do the most good, and produces a much more stable structure than one with a puddled core at the center. The down-stream portion of the structure, containing the coarser particles, will act, to a certain extent, as a drain for the

Mr. Justin. more impervious up-stream portion of the dam, and the line of saturation will enter the foundation at a point much farther up stream than would be the case if the usual central pool were used.

By wasting the finest particles a considerable yardage would be sacrificed, but as their inclusion in the structure has been shown to be a source of danger, it is surely the part of true economy to omit them. Also, these rejected fines are not entirely wasted. Flowing up stream from the dam, they will be deposited and tend to fill the interstices of the natural ground surface and thus decrease the flow of water through the soil under the dam.

There are doubtless certain materials, the effective sizes of which are so small that this method would not be applicable but, in that case, it is the part of wisdom to use some method other than hydraulic sluicing for construction.

Mr. Hazen. ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).—The writer is gratified to have had nineteen engineers, representing the widest experience in dam construction, take part in the discussion of his paper. The interest thus shown is evidence of the importance of the questions raised, and the writer feels that this broad and representative discussion will do much to clear them up. As these discussions, for the most part, consist of additional data and new points of view not inconsistent with the propositions which the writer has presented, he will not attempt to review them in detail, but will refer only to a few matters.

Some of the discussions show a misconception of the conditions at the Calaveras Dam prior to the slip. Without attempting a complete statement, it may be said that no puddle-core material found an outlet through the dry fill to either face of the Calaveras Dam at any time. The material in the up-stream toe which slipped was placed in horizontal layers only, and not on slopes. The lower toe, which was thinner and which did not slip, was placed in part by dumping on slopes. Hence, so far as the placing of material in this manner tended to weaken the embankment, it would have weakened the lower toe; and as this toe did not fail, certainly the manner of placing played no part in the actual slip.

At the time of the slip the concrete paving on the upper slope of the dam extended only to Elevation 663, while the top of the dam was at Elevation 770. The concrete paving was, therefore, 107 ft. below the top of the embankment and can have had very little influence on the conditions that led to the slip.

The water surface in the lake at the time of the slip was 116 ft. below the top of the dam, and the difference in elevation between the pool in the center of the dam and the lake was certainly sufficient, so that no retardation of drainage can be assumed to have resulted from back pressure.

* New York City.

Mr. Hazen.

It must be remembered also that water had been stored for three seasons before the slip, and that a year previous thereto it had been 20 ft. higher than it was at the time of the slip.

In regard to cost, it may be interesting to note that the company's records show that the actual direct cost of sluicing about 1 500 000 cu. yd. of material to the dam was 26.8 cents per cu. yd. When the costs of camps, roads, engineering, interest during construction, and all other expenses are included, the total cost was 34.6 cents per cu. yd. This, of course, gives no indication of what prices would be in 1920, but the writer believes that neither at that time, nor at the present time, could dry fill be placed at a price comparable with that at which the material was sluiced.

Mr. Holmes' description of the dams for which he has acted as Resident Engineer, with the analyses of materials and tests of permeability, are of the greatest interest. The core has been handled so that the grain size in some cases, on the analyses submitted, is considerably above the limit suggested by the writer; and not only this, but the record of Mr. Holmes' tests on permeability which he has given the writer in greater detail than in the discussion, show that the rate of seepage and the actual leakage through this coarse core material are not greater than those computed by the method used by the writer, but, instead, are somewhat smaller. In other words, there is complete demonstration that core material of this relatively coarse grain size is, for practical purposes, sufficiently water-tight.

Messrs. Burr, Henny, and Binckley refer to the fact that the dividing line between the core and the toe is not vertical, but on an incline, and that there is an upward component of the pressures of the core on the toe, which tends to lift the toe and reduce its resistance to sliding. This matter is more important as the core is wider. It was important at Calaveras. It would not be important for a much narrower core. Mr. Henny contributes an interesting calculation on the reduction in stability growing out of the presence of water back of the dam at various elevations.

Mr. Dunham also suggests that the Calaveras Dam probably slipped on an inclined surface rather than a horizontal one, and the writer believes this surmise is correct. Near the center line of the dam it was clear that the line of slippage was approximately 60 ft. above the bottom of the valley. At the base of the upper toe all the materials slipped (including the gravel from the creek bed that had been placed hydraulically), extending clear down to the natural surface and perhaps below it. These two points are known definitely, otherwise the slipping surface cannot be located, and numerous borings made after the dam slipped did not afford a basis for locating it. The material found in the borings was just the same whether it had slipped or not, and although the borings were made with a view to determining the position

Mr. Hazen.

of the surface of slip, the most careful attention failed to show any indications of it. The surface on which the Calaveras slip occurred, therefore, remains, and must remain, except for the two points mentioned, a matter of conjecture. Mr. Dunham is right, however, in saying that it was not horizontal.

All these matters are of importance in considering the stability of dams. How adequately to take them into account in future plans is the problem that is presented.

Reference has been made to the tests of frictional resistance of the materials at San Pablo. The writer hoped that G. W. Hawley, M. Am. Soc. C. E., Resident Engineer of the San Pablo Dam, who built the apparatus and made the tests, would present a description of them and a statement of the results obtained. As he has not done this, the record of a few results of the first tests made by Mr. Hawley, taken from the writer's notebook, is given in Table 7.

TABLE 7.

Direct pressure, in pounds per square inch.	Approximate depth of fill, in feet, to produce this pressure.	Lateral Pressure, in Terms of Direct Pressure Required to Displace Moist Material.		
		Orindian blue shale.	Monterey brown shale.	Worn brown shale, sand and small stones from dam.
17.4	21	0.67	0.78	0.57
34.9	42	0.54	0.67	0.54
52.4	63	0.46	0.61	0.55
69.8	84	0.45	0.55	0.57
87.3	105	0.45	0.53	0.56
104.7	126	0.45	0.50	0.56
122.2	147	0.45	0.50	0.56
139.6	168	0.45	0.49	0.50
157.1	189	0.44	0.48	0.54
174.6	210	0.45	0.46	0.54
Approximate average for results with pressures corresponding to more than 100-ft. depth of fill		0.45	0.49	0.54

Each figure in Table 7 represents the mean between two observed values. The first was of the pressure required to start the movement; that is, the maximum pressure observed before there was a slip; and the second was the pressure required to keep the slip moving after it was once started. In a general way, the latter pressure was about 10% less than the former. In other words, the friction required to start movement was 5% greater than that indicated by the figures, while the friction after movement was 5% less.

The writer believes that this kind of experimentation offers promise of better knowledge of different materials, and of the effect of varying

Mr. Hazen.

quantities of water in the same material; and he hopes that a complete record of the San Pablo tests may be published, and tests of other materials made.

Answering Mr. Dunham's questions, the section of the dam for which he asks may be found in *Engineering News-Record*, of December 26th, 1918, and a summary of the data relating to the slip may be found there and also in the issue of April 4th, 1918.

Mr. Morgan thinks that a clear distinction should be made between the compacting of the material by drainage and that by consolidation through settling under water. It seems to the writer that such a distinction may be hardly practicable, and he is inclined to include the settling through water (which necessarily involves an upward flow of the water through the material) under the term drainage.

The little volcanoes at the top of soft core material, which Mr. Morgan has mentioned, have been observed by the writer, who believes that they probably occur only in material with a large percentage of voids, and that they do contribute largely to the rapid first consolidation of material. Later, the material becomes more solid, and this volcano action stops. It must stop before the material has acquired any useful degree of stability, and for this reason the volcano action cannot be an element of importance in the final consolidation. It may be that it accounts for the original consolidation of the material to approximately 50% of voids in the course of a few months, which would otherwise be difficult to explain.

There can be no doubt that drainage of saturated material is essential to consolidation. The core material is deposited under water. Its voids are completely filled with water. Consolidation can take place only as the surplus water is excluded.

The conditions of flow are essentially the same whether the movement of the water is vertical or horizontal. It may be that the hydraulic slope for the vertical flow will be greater than was mentioned in the paper. This proposed value did refer to horizontal flow. For vertical flow the hydraulic slope may be taken as the specific gravity of the material less 1.00. A material with 50% of voids has a specific gravity wet of 1.825, and the hydraulic slope for upward flow is therefore 0.825. Using an estimated rate of seepage based on this value would change the figure, but not the essential conditions as described.

How to get the excess water out of the core material is the one dominant problem with fine-grained core material. It is the water that holds the particles apart and prevents consolidation; if it can be taken out by drainage, consolidation follows as a matter of course and to the full extent that water is removed.

In considering experience with shallower materials elsewhere, evaporation from the surface and bringing water up from below by capillarity is often a dominant factor, and this must be kept in mind in making comparisons with observations obtained under such conditions.

Mr. Hazen.

Mr. Paul thinks that the writer's descriptions of 50% material are inconsistent in that at one time it is described as being capable of standing up in a boring driven beyond the casing for several feet, and in another paragraph as exerting full lateral pressure. The explanation of the apparent discrepancy lies in the cohesion of the material. Cohesion is real and important. It is cohesion that holds the vertical sides of a cut in such material to a depth of several feet. It is cohesion that gives it the firmness of stiff putty. It is cohesion that holds the particles together as an arch about the borings below the end of the casing. Cohesion exists equally in a great mass of material under heavy pressure, but there it becomes so small in comparison with the pressure, that it is no longer able to maintain stability and the material becomes capable of flowing.

The depths of penetration mentioned by Mr. Paul, as observed at Calaveras, are understood by the writer to be maximum depths at special points at a time long before the slip. Before the slip occurred there had been a temporary cessation of operations, and the core material had consolidated and hardened to a point where penetration was resisted at a depth of a very few feet; and so far as this kind of evidence goes, the core material at the Calaveras Dam at the time of the slip was satisfactory.

The one great lesson to be learned from the Calaveras slip is that material consolidated to this extent and having the consistency of stiff putty, capable of resisting penetration is, nevertheless, capable of flowing in large masses.

Another question in which the same line of thought is involved is whether deductions from readings of small and very ingenious pressure gauges may be safely used in calculations of pressure in large masses. The writer is in doubt on this question, and is disposed to proceed slowly pending further demonstration. The question which he raises is not as to the accuracy of the record of the gauges (of which he has no knowledge), but whether, in view of cohesion, actual pressures on very small areas really indicate conditions in large masses. For instance, would the pressure gauge against the surface of material standing beyond the casing of a bore hole show full lateral pressure? The writer is in no doubt of the fact that some material that will stand in that way without support is, nevertheless, capable of flowing almost like water when the mass is sufficiently great. Mr. Goldbeck's gauges seem to be ingenious devices, and the results obtained by their use in actual dams will be studied with interest.

Mr. Muckleston's observations suggest the possibility of chemical treatment to improve the physical condition of colloidal clay. This certainly opens interesting possibilities.

Mr. O'Shaughnessy suggests reducing the flow line of the Calaveras Reservoir 37 ft., in the interest of greater safety. Reducing the flow line by this amount would reduce the capacity of the reservoir by about

19 300 000 000 gal., or by about 40 per cent. The full proposed capacity is not too much to hold the waters of floods that occur only at intervals of several years, and make them available at all times, especially during the dry years sometimes experienced in California. If the flow line should be lowered as proposed, it would reduce the maintainable supply through a series of years by 10 000 000 gal. per day. In the San Francisco District, 10 000 000 gal. of water per day are worth a certain number of millions of dollars. To reduce the height of the dam, and with it the capacity and value of the source, should not be done unless there is valid reason for believing that a dam built to the full height cannot be made safe. Mr. Hazen.

Mr. Binckley's statement in regard to Mr. Schuyler's later views and experience, unfortunately not expressed in writing before Mr. Schuyler's death, are most interesting and helpful; and it is gratifying to the writer that the views which he has presented are supported in this manner.